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PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

PLEASE RETURN

BEAVER CREEK AND BUFFALO CREEK BASINS
CHESSMAN MAIN DAM AND CHESSMAN SADDLE DAM
NEAR RIMINI, MONTANA
MT-1090 AND MT-1562

PREPARED FOR:

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PHASE I INSPECTION REPORT

NATIONAL DAM SAFETY PROGRAM

BEAVER CREEK AND BUFFALO CREEK BASINS

CHESSMAN MAIN DAM AND CHESSMAN SADDLE DAM

NEAR RIMINI, MONTANA

MT-1090 AND MT-1562

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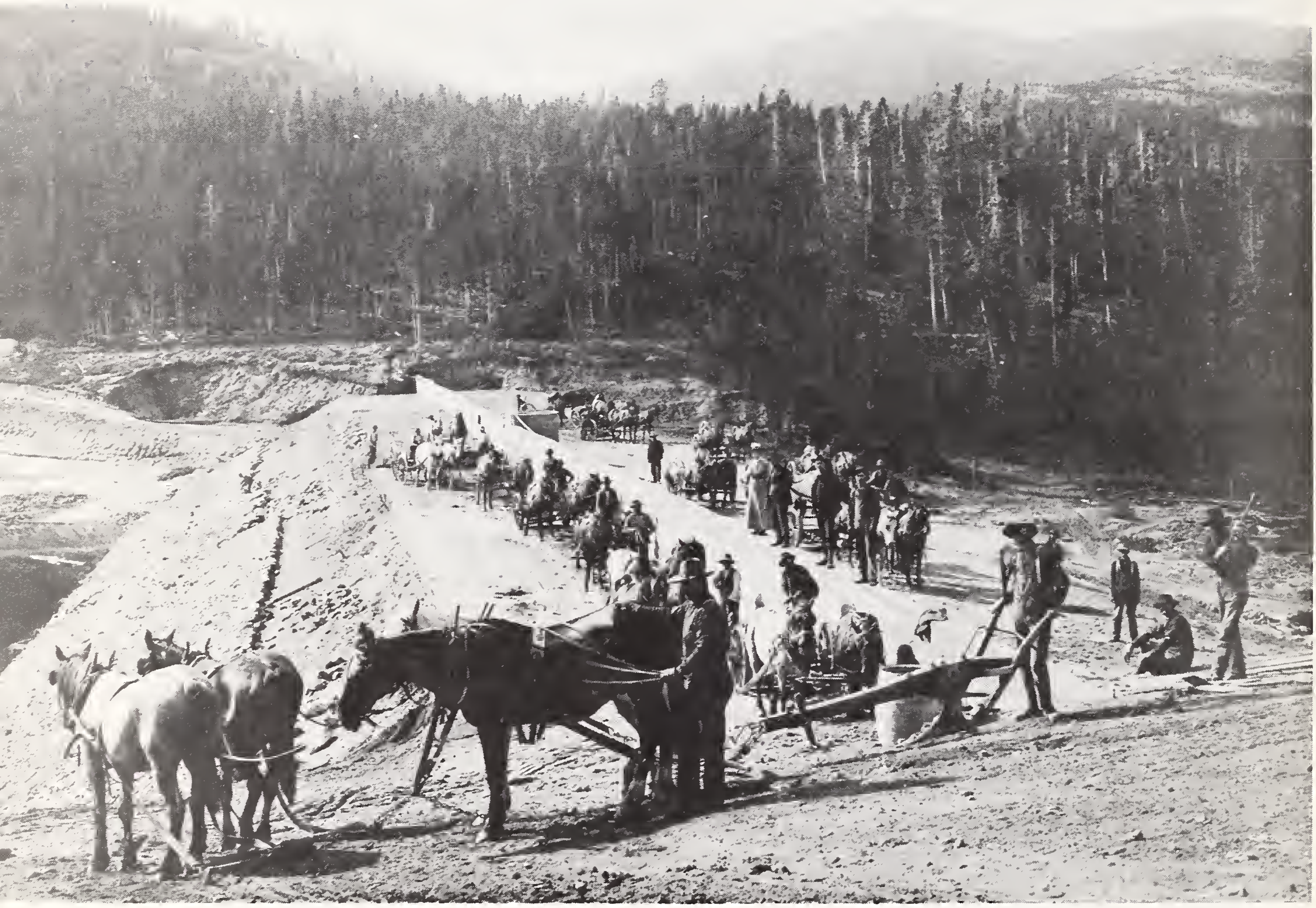

CHESSMAN MAIN DAM DURING CONSTRUCTION
PHOTO COURTESY OF CITY OF HELENA

EXECUTIVE SUMMARY

CHESSMAN DAM AND CHESSMAN SADDLE DAM

INTRODUCTION

Morrison-Maierle, Inc., under contract with the Montana Department of Natural Resources and Conservation (DNR & C), who are under contract with Seattle District, U.S. Army Corps of Engineers, inspected Chessman Main Dam and Chessman Saddle Dam on August 9, 1978 under authority of Public Law 92-367 and in accordance with the "Recommended Guidelines for Safety Inspection of Dams" (Reference 9) prepared by the U.S. Department of the Army, Office of the Chief of Engineers. Chessman Reservoir is located in Helena National Forest approximately 12 miles southwest of the City of Helena, Montana on Beaver Creek in the upper Ten Mile Creek drainage (Plate 1). The project is owned and operated by the City of Helena as a municipal water supply facility under a Forest Service special use permit. It is part of the city's "Ten Mile" water supply system and augments supplies from Ten Mile Creek. All elevations referred to in this report are relative to project datum.

FINDINGS AND EVALUATION

Chessman Reservoir is formed by two earthfill dams; Chessman Main Dam across Beaver Creek and Chessman Saddle Dam located north of the main dam on the divide with Buffalo Creek. Chessman Main Dam is a 40-foot-high 440-foot-long embankment with a crest elevation of 105 feet. A concrete core wall extends the entire length of the main embankment. Regulatory control for water supply releases is provided by a valved conduit located in the main dam which has a capacity of approximately 30 cubic feet per second (cfs). Chessman Saddle Dam, located 2,600 feet north of the main dam, is 17 feet high and 105 feet long with a crest elevation of 101 feet (4 feet lower than the main dam). A concrete core wall extends the entire length of this embankment also and is exposed along the dam crest at an elevation of 101 feet. The 10-foot breadth of the saddle dam crest from the core wall downstream is paved with concrete. The project spillway is located in the saddle dam.

As the project exists, a roadfill extends across the approach bay to the spillway to allow access along the east shore of the reservoir. The road was built to replace a bridge which originally crossed the spillway. The road is at an elevation of 101 feet (equal to the saddle dam crest) and essentially prohibits the spillway from functioning until the reservoir exceeds that elevation.

At normal operating pool, elevation 99 feet, the project impounds 1,630 acre-feet of water but is capable of impounding 1,870 acre-feet at the top of the saddle dam (elevation 101 feet). Inspection criteria classify both dams as intermediate in size and, because of possible downstream effects of failure, the downstream hazard potential is high (category 1). Inspection criteria recommend such projects to safely handle a full Probable Maximum Flood (PMF). The PMF is the flood expected from the most severe combination of critical meteorologic and hydrologic conditions reasonably possible in the watershed. The PMF estimated for this study was assumed to occur in the summer as a result of rainfall from a 6-hour thunderstorm. A total storm precipitation of 10 inches applied to the 1.33-square-mile drainage yields a PMF with a peak inflow of 6,635 cfs and a volume of 660 acre-feet.

A routing of the estimated PMF was performed for two conditions: 1) with the roadfill in place; and 2) with the roadfill removed. The analysis of the first or existing condition assumes the pool to be at the top of the road (elevation 101 feet) with flashboards removed from the spillway and the spillway fully operational when the PMF begins. The roadfill would be immediately overtopped and presumably washed out. The 590 cfs capacity of the spillway is inadequate and would be rapidly exceeded causing the saddle dam to be overtopped by a flood derived by taking 30 percent of the full PMF ordinates. The reservoir would rise to elevation 103.5 feet (1.5 feet below the top of the main dam) during the PMF. Even though the saddle dam crest is paved, the overtopping conditions and the nature of the embankment material would result in failure of the structure causing potential downstream flooding, property destruction, and possible loss of life as the surge traveled down Buffalo Creek and Lump Gulch.

A second routing of the PMF was performed with original spillway design conditions; that is, with the roadfill removed, the spillway fully operational without flashboards, and the pool at the spillway crest elevation of 96 feet when the PMF begins. Under these conditions the reservoir would rise to elevation 101.1 feet, 0.1 foot over the top of the saddle dam. This routing suggests the project might safely handle the PMF if restored to original design conditions.

Even though overtopping of the main dam is not a potential threat, conditions were observed which cause some concern for the stability of the main dam embankment. With the reservoir at normal pool elevation, an apparent seepage gradient of 4.7 feet exists between the reservoir and the core wall of the main dam, as was observed in the valve chamber. This indicates a significant flow is occurring through the embankment and either over, under, through, or around the core wall. No surface seepage was observed during the inspection; however, zoned plant growth suggests the presence of near-surface seepage and the reservoir operator reports that at pool elevations higher than 99 feet he has observed seepage at the toe of the embankment. While no major slope failures are evident, minor displacements suggest that stability may be marginal under maximum pool conditions. Additional information is needed to properly evaluate stability of the main dam. The saddle dam stability appears adequate; however, observed conditions and available information were inadequate to support conclusions regarding seepage and stability conditions.

RECOMMENDATIONS

Recommendations made in this report need to be acted upon without delay. The following steps are recommended for immediate implementation to reduce the immediate risk and to develop permanent corrective measures:

1. Develop, implement, and periodically test an emergency downstream warning plan for the areas below each dam.

2. Remove the road berm across the spillway approach bay and restore the spillway structure to full serviceability and maintain the pool at maximum elevation of 96 feet project datum.

3. Conduct more detailed hydrologic and hydraulic routing studies below both dams to better determine the downstream hazards and required spillway capacity and modify the project as indicated.

4. Conduct seepage and stability analyses including an evaluation of the project's response to a seismic event.

Prior to implementing any report recommendations the owner is urged to coordinate with the Montana Department of Natural Resources and Conservation.

Morrison-Maierle, Inc.

Rodger C Foster

Rodger C. Foster, P.E.
Project Manager

PERTINENT DATA

CHESSMAN DAM, MT-1090; CHESSMAN SADDLE DAM, MT-1562

LOCATION:	Township 8 North, Range 5 West Section 2 latitude 112°11', longitude 46°28' Lewis and Clark County, Montana
WATERSHEDS:	Beaver Creek, Buffalo Creek
OWNER/OPERATOR:	City of Helena, Montana
PURPOSE:	Municipal Water Supply
YEAR CONSTRUCTED:	1908
CHESSMAN DAM:	Earthfill dam with concrete core wall
SADDLE DAM:	Earthfill dam with concrete core wall Project spillway located in the dam
MAIN DAM CLASSIFICATION:	Intermediate size, high downstream hazard potential
SADDLE DAM CLASSIFICATION:	Intermediate size, significant downstream hazard potential
KEY ELEVATIONS: (Project Datum)	
Top of main dam	Elevation 105 feet
Top of main dam core wall (not verified)	101 feet
Top of saddle dam and core wall	101 feet
Top of spillway road berm	100 feet
Maximum operating pool (30 feet gage height)	100 feet
Normal operating pool (29 feet gage height)	99 feet
Spillway Crest	96 feet
Outlet at toe of main dam	65 feet

MAIN DAM DIMENSIONS:

Hydraulic Height	40 feet from outlet
Structural Height	57 feet from bottom of core wall
Crest Length	440 feet
Crest Breadth	15-17 feet
Face Slopes	1V on 2H

SADDLE DAM DIMENSIONS:

Hydraulic Height	17 feet from toe of chute
Structural Height	27 feet from bottom of core wall
Total Length	105 feet

SADDLE DAM DIMENSIONS (Continued):

Crest Breadth	10 feet, concrete paved
Face Slopes	Upstream 1V on 1.5 H Downstream 1V on 2.5H

SPILLWAY:

Location	Left center of saddle dam
Type	3 bay concrete-lined chute with trapezoidal shaped broad crested weir control. Bays formed in concrete core wall.
Width	22.5 feet (total opening of 3 bays)
Control	Provisions for flashboards to elevation 101 feet. Berm upstream at elev. 101 feet blocks spillway approach.
Crest Elevation	96 feet
Chute Slope	IV on 3H
Capacity at Overtopping	590 cfs, elevation 101 feet

OUTLET WORKS:
(Main Dam)

Type	Submerged 16-inch-diameter Cast Iron inlet pipe discharging to a 24-inch by 27-inch concrete arch conduit.
Length	100 feet CI pipe 88 feet concrete arch conduit
Control	Gate valve located in dry-well valve chamber at center of dam.
Capacity	Approximately 25 to 30 cfs at normal pool.

RESERVOIR:

Surface Area	- 114 acres @ Normal Pool
Storage at:	
Top of Main Dam	- 2370 acre-feet @ elev. 105 feet
Top of Saddle Dam	- 1870 acre-feet @ elev. 101 feet 1400 acre-feet active Storage
Normal Pool	- 1630 acre-feet @ elev. 99 feet
Spillway Crest	- 1330 acre-feet @ elev. 96 feet

CHAPTER 1

BACKGROUND

1.1 INTRODUCTION

1.1.1 Authority

This report summarizes the Phase 1 technical inspection and evaluation of the Chessman Main Dam and Saddle Dam, a non-Federal project owned by the City of Helena, Montana. Authorization for the inspection is contained in the National Dam Inspection Act, Public Law 92-367, dated 8 August 1972. Nothing contained in this Act and no action or failure to act under this Act shall be construed (1) to create any liability to the United States or its officers or employees for the recovery of damages caused by such action or failure to act; or (2) to relieve an owner or operator of a dam of the legal duties, obligations, or liabilities incident to the ownership or operation of the dam.

1.1.2 Purpose

The purpose of the inspection and evaluation of Chessman Dam and Chessman Saddle Dam is to determine if conditions exist which constitute a danger to human life or property so that timely corrective measures can be taken by non-Federal interests.

1.1.3 Inspection

The findings and recommendations presented in this report are based upon an on-site inspection of the project conducted 9 August 1978, review of construction photographs and as-built drawings, interviews with the reservoir operator, the Public Utilities Director for the City of Helena, and the forest engineer who performed previous dam maintenance inspections, and files of the Helena National Forest, including maintenance inspection reports. The safety inspection and evaluation were performed in accordance with the "Recommended Guidelines for Safety Inspection of Dams", Department of the Army, Office of the Chief of Engineers.

Morrison-Maierle, Inc. personnel who conducted the field inspection were:

Rodger C. Foster - Water Resource Engineer, Team Leader
William Keith - Structural Engineer
Michael Kaczmarek - Geologist
Kenneth Wiegand - Geologist

Others present during the inspection included:
Captain Donald E. Warner - U.S. Army Corps of Engineers,
Assistant Project Manager,
Dam Safety Program,
Seattle District

Richard Nisbet - Director of Public Utilities, City of Helena;
Ed Loranz - Reservoir Operator, City of Helena, Water Department;
Glen McDonald - Montana Department of Natural Resources and Conservation;
Paul Nickerson - U.S. Forest Service, Missoula, Montana;
Ron Hagan - Supervising C.E., Helena National Forest.

1.2 DESCRIPTION OF PROJECT

1.2.1 General

Chessman Reservoir is a municipal water supply storage project owned and operated by the City of Helena, Montana. It is located in Helena National Forest in Section 2, Township 8 North, Range 5 West, in Lewis and Clark County about 12 miles southwest of the City of Helena. The reservoir is perched atop a divide at the head of Beaver Creek in the upper Ten Mile Creek drainage about 3.2 miles above the community of Rimini (Plate 1). Built in 1908, the reservoir is actually formed by two earthfill dams, a main embankment situated at the west end of the reservoir across Beaver Creek, and a smaller saddle dam situated 2,600 feet to the north on the divide between Beaver Creek and Buffalo Creek. Several photographs, maps, and drawings describing the project are presented following the text of this report.

Chessman Main Dam, is 40 feet high, 440 feet long and impounds 1,630 acre-feet in 114 surface acres at a normal pool elevation of 99 feet (project datum). The height of the dam, impoundment volume of the reservoir, and the residential development in Rimini and along Ten Mile Creek cause Chessman Main Dam to be classified as being of intermediate size with a high downstream hazard potential.

Chessman Saddle Dam is separated from the main embankment and contains the spillway structure for the project. The saddle dam is situated in a low divide separating the Beaver Creek drainage, in which the reservoir is located, from the Buffalo Creek drainage. Prior to construction of the project, no natural flow existed across the drainage divide. However, with construction of the dams and the creation of the reservoir, spillage of excess water was diverted to Buffalo Creek via the saddle dam spillway. The dam is 17 feet high and 105 feet long, with a spillway formed into the core wall. The total active storage impounded by the saddle dam is approximately 1,400 acre-feet.

Downstream of the spillway on Lump Gulch there are possibly three dwellings which might be affected by dam-failure surge flow. Below the confluence of Buffalo Creek and Lump Gulch there is sparse but increasing development along the stream extending to the confluence with Prickly Pear Creek near the community of Clancy. The impoundment volume causes Chessman Saddle Dam to be classified as being of intermediate size and with three dwellings that might be affected by failure, the downstream hazard potential is high in accordance with Corps of Engineers guidelines.

1.2.2 Regional Geology

Chessman Dam and Reservoir are located within the geologic province of the Boulder batholith. Boulder batholith is a generally north to south aligned composite mass of intrusive igneous rock approximately 70 miles long and 35 miles wide with its northern boundary located near Helena, Montana. The thickness of the batholith is measured in terms of miles.

The predominant rock types comprising the bulk of the batholith are quartz monzonite and granodiorite. These rock types are very similar to granite in appearance, weathering, and engineering characteristics and differ from granite only in the ratio of two types of feldspar minerals present in the rock. Quartz monzonite and granodiorite in massive monolithic outcrops are exposed along ridges and valley wall cliffs. Forested hillsides with up to 50 percent slope are typically covered with an average 10-foot thickness of weathered residuum and colluvial slope deposits, consisting of silty to clayey, coarse and granular, loose to friable sand which are SM and SC soils under the Unified Soils Classification system.

The quartz monzonite and granodiorite masses are intruded in turn by younger, smaller bodies of finely crystalline aplite, alaskite, quartz latite, and rhyolite as well as vein fillings of quartz, chalcedony, and some pegmatites. These volcanic and vein rocks exert localized influence on gross engineering characteristics and requirements of specific sites; however, more engineering problems result from joint and fault structures which constitute weaknesses in the batholithic rocks.

The main mass of the Boulder batholith is faulted by at least three families of normal faults, each having a distinctive alignment and trend. Displacement along many of the faults is only a few feet; however, many of the larger faults have displacements of tens to hundreds of feet. Typically, rocks along the larger faults are sheared and altered and weathering may extend to depths of 30 feet or more. Joint densities are often greater near the larger faults. All of these factors result in structurally weaker shear strengths and lower foundation bearing capacities in the fault zones.

1.2.3 Seismicity

The Chessman project is located in seismic probability Zone 3. The seismic probabilities used herein divide the United States into four seismic risk zones based on the record of the severity of ground shaking and the reasonable expectancy of earthquake damage. Seismic probability or risk in Zone 3 indicates that major destructive earth-quakes may occur. Zone 3 corresponds to a potential intensity of VIII or higher on the Modified Mercalli intensity scale of 1931.

Chessman Dam and the surrounding area near Helena, Montana, has an extensive history of earthquake activity. The first record is of an intensity VI shock in Helena on May 22, 1869. In 1872, intensity VI shocks occurred at Helena on December 10 and December 11. In October and November of 1935, a series of 1,347 shocks and aftershocks, ranging from maximum intensities of V to X and including magnitude 6 and magnitude 6.25 earthquakes (Richter magnitude scale), rocked the Helena area causing four deaths, several injuries, and property damage exceeding $4 million.

Severe earthquake shocks could affect Chessman Dam in two possible ways: (1) by embankment settlement or failure of the side slopes; or (2) by tensile cracking of concrete in the core wall or spillway structure due to shaking or foundation displacement. The potential for failure of the earthfill embankment is discussed in Section 2.4.3 of this report.

The potential for displacement of the foundation materials appears to be minimal. No evidence of a fault or fault trend is present at or near the dam and faults in the Boulder batholith do not display evidence of recent movement. The so-called shear zone shown in the right abutment on Plate 5 is a zone of hydrothermally altered rock in place that does not appear to be granulated or displaced.

No records of damage to the dam or spillway structure during the 1935 earthquakes have been found. Evidently, the concrete core walls and earthfill embankments withstood the seismic loads without failure. If fractures were caused in the main dam core wall by the seismic shocks, the amount of seepage through the core wall has been within the hydraulic capacity of the foundation and dam embankment to convey the seepage away from the structure in the subsurface under historic pool conditions.

1.2.4 Site Geology

The local geology at Chessman Reservoir and dams is depicted on Plate 5. The geologic map by Becraft et al (1963) (reference 1) shows the terrain underlying and contiguous to the dam and reservoir to be supported by quartz monzonite and granodiorite bedrock. A number of northeast trending chalcedony veins and associated hydrothermally altered zones are present in the vicinity of the dam and reservoir. A small northwest trending strike-slip fault is present about one-half mile south of the reservoir and is the only fault evident.

1.2.5 Description of Dam and Appurtenances

1.2.5.1 Chessman Main Dam

Chessman Main Dam is an earth embankment with a concrete core wall extending the full length but not the full height of the dam. Field measurements taken during the inspection verify the as-built dimensions (Plate 3) as being 440 feet long, 40 feet high from the streambed to the crest, and 15 to 17 feet wide at the crest. The only outlet through the dam consists of a valved pipe used to regulate releases to the City of Helena's water distribution system.

A dry well valve chamber provides access to the control valve at the center of the dam. Drawings and photographs of Chessman Main Dam are provided following the text of this report and include photos 1 through 19 and plates 2, 3 and 5.

1.2.5.2 Chessman Saddle Dam

Chessman Saddle Dam is an earth embankment 17 feet high and 105 feet long with a concrete core wall extending the full length. The core wall is exposed along the upstream edge of the crest of the embankment and the full 10-foot width of the crest is paved with concrete (Photos 35 and 36). The spillway is formed into the core wall and consists of three bays with stoplogs. A riprapped and paved chute 30 feet long directs the flow to heavily overgrown Buffalo Creek channel.

At one time a wooden bridge crossed the spillway to provide access along the east shore of the reservoir. After the bridge had deteriorated beyond repair, it was removed and substituted with a roadfill constructed about 200 feet upstream across the approach channel to the spillway. The road, with an elevation approximately equal to the top of the saddle dam prohibits the spillway from functioning as designed and has an important bearing on the safety of the project. Details of the saddle dam and spillway are presented in Photographs 26 through 35 and Plates 2, 3, and 5.

1.2.5.3 Beaver Creek Road Berm

A road berm was constructed several years ago at the south end of the reservoir which crosses the inlet bay where Beaver Creek flows into the reservoir (Photos 24 and 25). The road reportedly was constructed to contain debris originating from beaver activity in that portion of the drainage and to contain heavy algae blooms and aquatic growth which occur in the shallows of the inlet. The top of the road is at an average elevation of 103.5 feet (project datum) with adjacent water levels equalizing through drainage provided by two 18-inch-diameter corrugated metal pipe (CMP) culverts and a valved 10-inch-diameter cast iron (CI) pipe. The road material is highly erodible and the ponding area behind the road has no significant effect on the routing of high flows into the reservoir.

1.2.5.4 Red Mountain Ditch

A water supply aqueduct known as the Red Mountain Ditch supplies water from the Bonner Creek drainage to Chessman Reservoir. The ditch extends about 7½ miles from the southwest end of the reservoir near the dam, around Red Mountain, intercepting the runoff from Sally Bell Creek. Capacity of the ditch is about 15 cfs and any excess flow will spill into the Beaver Creek drainage below Chessman Dam. Photographs of the aqueduct are presented following the text of this report at Photos 20 through 23.

1.2.6 Design and Construction History

Chessman Main Dam and Chessman Saddle Dam were designed by Eugene Carroll for the Helena Water Works Company in January 1907. Construction is thought to have been completed in 1908 although no documentation of that fact has been found. The only documentation found of the design and construction of the dams consists of one sheet labeled "Plan of Dam at the Chessman Reservoir", dated January, 1907 which is also noted "as-used" (Plate 3), and some historical photographs of the construction (Photos 3, 4, 5, 7, 26). No information is available as to site investigations, material testing, construction inspection, borrow areas, or toe drains as related to original construction.

Inspection reports from the files of Helena National Forest mention repairs or modifications to the dam which may have been made in the period 1933 to 1935. Inspection of the dam revealed additional fill at the outlet, and drains at the outlet and downstream of the outlet which are modifications not shown on the as-used drawings. No documentation has been found concerning any repair work or modifications to the dam after 1935 nor is there any information concerning damage to the structure resulting from the earthquake.

CHAPTER 2

INSPECTION AND RECORDS EVALUATION

2.1 SPILLWAY DESIGN FLOOD

Chessman Main Dam and Chessman Saddle Dam impound over 1,000 acre-feet of water and are classified as intermediate sized dams. Based on a visual assessment each dam is further classified as having a high downstream hazard potential due to downstream development. Inspection criteria recommend the spillway design flood (SDF) for high hazard, intermediate sized dams to be a full probable maximum flood (PMF).

2.2 HYDRAULICS AND STRUCTURES

2.2.1 Spillway

The spillway for Chessman Reservoir is located in an earthfill saddle dam separate from and 2,600 feet to the north of Chessman Main Dam. The embankment is 17 feet high and 105 feet long, with a concrete core wall extending the entire length. The spillway section is formed by a 24-foot-wide by 5-foot-high notch in the core wall located approximately 25 feet from the west abutment. The spillway is a trapezoidal broad-crested weir with a breadth of 10 feet, an approach slope of 1 vertical (V) on 2 horizontal (H) and a chute slope of 1V on 3H. Control is provided by three bays of stoplogs spaced equally in the 24-foot total spillway width with a total clear opening of 22.5 feet. The concrete chute narrows slightly from a floor width at the entrance of 24 feet to a width at the toe of the chute of 23.3 feet in a length of 30 feet. The training walls taper from a width of 34 feet and a height of 5 feet at the crest of the spillway to zero height where the walls meet the chute floor at the toe.

The spillway chute has a concrete floor and side walls and much of the chute floor is covered with sediment and sod to a depth of 4 to 6 inches (Photo 27). A few bush-size trees are growing through the concrete pavement in the chute. Upstream of the spillway, brush and dead fall have accumulated along the southeast bank of the approach channel which could cause a debris problem at the spillway.

The spillway has not been operated in several years and the stoplog supports are in a deteriorated condition. The intermediate support posts and stoplog guides are anchored in the core wall and are sound, although very rusty. A few deteriorated stoplogs are in place but the remainder are missing. Sand and sediment have drifted to the top of the existing stoplogs. There is no means for removing stoplogs. Photographs 26 through 34 show the condition of the spillway.

The spillway approach channel is a natural bay approximately 250 feet long and transitions from approximately 250 feet wide at the reservoir to 100 feet at the saddle dam. A road berm crosses the approach channel about 200 feet upstream from the dam at an approximate elevation of 101 feet which is equal to the crest of the saddle dam. The road is 2 feet above normal reservoir elevation and provides additional storage capacity sufficient to store a flood volume approximately equal to one-third of the PMF.

A pond of water exists between the road berm and the spillway and, on the date of inspection was at approximate elevation of 96 feet (spillway crest elevation) or 2.1 feet lower than the reservoir. Due to stoplogs and sediment build-up in the spillway there was no water being spilled. Photographs 28 through 31 show the ponded area behind the spillway.

Inspection reports from the Helena National Forest files indicate that an 18-inch-diameter culvert passes through the road although none was found by this inspection. As evidenced by a 2-foot difference in water surface across the road, the berm functions as a dam and controls flow to the spillway. Because of the additional available storage behind the road, only relatively rare flood events will overtop the road and pass the spillway.

A stage-discharge curve (Plate 4) was developed for the spillway and for an overtopping condition at the saddle dam. In analyzing the spillway, it was assumed all flashboards were removed and the approach crest and chute were cleared of sediment and debris so that the full capacity of the spillway would be available. The spillway portion of the rating curve was developed using a standard broad-crested weir equation assuming a coefficient of 2.7 with effective length based upon end contractions of .2(Head). The computed discharge for the overtopping condition is also based on a standard broad-crested weir equation assuming a coefficient of 2.7. Since there are no end constrictions at the abutments, the effective weir length used in the overtopping analysis is 81 feet or equal to the total dam length less the width of the spillway.

2.2.2 Chessman Main Dam Outlet Works

The outlet works through the main dam provides the only means of regulating operational releases for the project. As shown on Plate 3, the outlet works consists of a 16-inch-diameter cast iron (CI) pipe encased in concrete which extends from the reservoir to a valve in a dry chamber located at the center of the dam on the downstream face of the core wall. A small portion of the pipe was observed where it was exposed in the valve chamber and appeared to be in good condition. Downstream of the valve chamber the 16-inch-diameter CI pipe discharges to a 24-inch by 27-inch concrete arch conduit which exits at the center toe of the dam.

The as-built drawings (Plate 3) indicate the reservoir inlet to the conduit is through a 20-inch-diameter riser pipe extending 5 feet above the invert of the outlet pipe. The cover plug valve indicated on the drawings is not used as an outlet control and was observed lying on the ground at the valve control house (Photo 19). The reservoir operator said he thought the inlet was modified and extended into the reservoir but has no personal knowledge of the modification. No documentation of any modification to the inlet could be found.

The valve chamber which provides access to the valve at the center of the dam is a dry well consisting of a semi-circular, concrete chamber rising approximately 40 feet from the flow line of the outlet conduit. The chamber is formed against the downstream face of the core wall. Access down the chamber is by ladders fastened to the original cast-in-place iron steps which are rusted to the point of unserviceability. Inspection of the valve chamber revealed no excessive infiltration and although water and some sediment in the chamber almost obscured the pipe from inspection, the water was draining from the chamber. A small one-inch-diameter hole in the downstream wall of the valve chamber 9 feet above the top of the pipe was observed. It had been plugged with a wood plug, apparently for several years, and was dripping water only a few drops per minute. Surface moisture was observed on the core wall face in the chamber beginning at an approximate elevation of 93 feet and extending to the bottom. Although there was no flow observed down the face of the wall, the moisture is significant in that it shows a substantial hydraulic gradient exists between the reservoir and the core wall. This would indicate there may be considerable seepage through and/or under the core wall. Further discussion of seepage conditions is presented in Section 2.4.2.2.

Atop the valve chamber is a circular valve control house (Photos 17, 18 and 19) which is normally locked and also serves as a storage house. The valve is operated from the house by a wheel connected to a valve stem. The valve was operated during the inspection and is in good working order.

The concrete arch conduit was too small to allow inspection except at the outlet and there was no deterioration, displacement, or damage observed. The channel downstream of the outlet is a narrow trapezoidal channel about 4 feet deep with a 3-foot-wide concrete bottom and steep riprap side slopes (approximately 3V on 1H). In several locations the side slopes have eroded from behind the riprap and the riprap is freestanding and self-supporting. In these areas riprap could easily collapse during high flows and block the channel. Some riprap has fallen into the mouth of the outlet pipe, partially restricting flow from the outlet.

At full pool the discharge capacity of the outlet works is estimated to be 25 to 30 cfs. Compared to the storage capacity of the reservoir and the magnitude of the PMF, any discharge through the outlet works would be insignificantly small and would not affect reservoir routing estimates. A rating curve for the outlet works was not determined for this analysis as it is assumed the outlet would not be operated in an extreme event.

2.2.3 Beaver Creek Road Berm

Beaver Creek road berm, located across the inlet of Beaver Creek to the reservoir, causes some restriction of normal flow to the main body of the reservoir and, consequently, increased ponding behind the road. The storage volume behind the road is small and the roadfill is highly erodable. The road would not have a significant effect on the hydrology or hydraulics of the reservoir system and would probably wash out when overtopped.

2.2.4 Freeboard

At a normal operating pool (99 feet) there is 6 feet of vertical distance to the crest of the main dam and 2 feet to the crest of the saddle dam. Under PMF conditions the main dam would have 1.5 feet of freeboard and the saddle dam would have no freeboard as it would be overtopped. Prevailing winds are generally from the west and wind-generated waves would be directed away from the main dam toward the saddle dam. With a fetch of approximately 2,000 feet wave heights would not be expected to exceed 1 to 1.5 feet. With wave runup being on the order of 2 to 3 feet, and under existing conditions, operating freeboard at normal pool is inadequate to prevent wave overtopping of the saddle dam.

2.3 HYDROLOGY

2.3.1 General Hydrologic Conditions

Chessman Reservoir, located at longitude 112° 11' and latitude 46° 28', is formed in the natural drainage of Beaver Creek about 5 miles east of the Continental Divide. At the normal operating pool (project datum elevation of 99 feet) the pool elevation would be approximately 6,212 feet MSL as estimated from the USGS Jefferson City 15 minute topographic quadrangle. Maximum elevation in the drainage is approximately 7,120 feet. The generally trapezoidal-shaped drainage of 1.33 square miles is approximately 2 miles by 1 mile in its largest dimensions with a longest stream length of 1.3 miles. The drainage is divided hydrologically into two basins consisting of the land area (1.15 sq. mi.) and the reservoir surface (0.18 sq. mi.). It is mostly forested with very little undergrowth and receives an average of 24 inches of precipitation annually in the form of rain and snow. An additional 5.4 square miles of drainage area contributes to the reservoir under normal operation through the Red

Mountain Ditch, which is a diversion aqueduct from the Banner Creek drainage. Inflow from the drainage is limited to the ditch capacity (15 cfs) since excess flow spills into Beaver Creek downstream of Chessman Dam. Because the contribution from the diversion is so small, the drainage and flow are neglected in the determination of the PMF.

2.3.2 Reservoir Storage and Spillway Discharge

Stage-storage relationships for Chessman Reservoir were provided to a stage elevation of 100 feet by the owner. That curve, extrapolated to an elevation of 105 feet at the top of the dam, is presented on Plate 4 along with a stage-discharge curve for the spillway.

2.3.3 Estimated Probable Maximum Flood (PMF)

In order to develop a probable maximum precipitation (PMP) for the Chessman reservoir drainage, a 6-hour summer thunderstorm event was selected as the most critical storm event. The PMP was obtained using procedures outlined in the U.S. Weather Bureau's, Technical Paper No. 38 (Reference 12). Using 5-minute precipitation time increments sequenced in a critical manner produced a total PMP for the 6-hour period of 10.0 inches. In calculating the probable maximum flood corresponding to the derived PMP, it was assumed that the Class B hydrologic soils (moderate infiltration) were saturated and would have a constant loss rate of 0.15 inches per hour. Unit hydrographs were developed separately for the watershed and the reservoir surface using procedures from the U.S. Bureau of Reclamation publication "Design of Small Dams" (Reference 13). The U.S. Army, Corps of Engineers, Flood Hydrograph Package Computer Program HEC-1 (Reference 8) was used to combine the PMP data with the unit hydrographs to develop a reservoir inflow hydrograph, or PMF. The PMF yields a peak flow of 6,635 cfs with a volume of 660 acre-feet.

2.3.4 Routing of the Estimated Probable Maximum Flood

2.3.4.1 Present Spillway Conditions

Routing of the probable maximum flood through the reservoir and the spillway was performed using the computer program HEC-1 as was the development of the flood hydrograph. In the routing analysis it was assumed the pool elevation at the beginning of the storm was at the top of the road which crosses the spillway approach channel at elevation 101 feet. It was further assumed that the full capacity of the spillway would be available and used when the road berm was overtopped and presumably washed out. Routing the PMF yields a peak outflow of 1,925 cfs which would cause the saddle dam to be overtopped by 2.5 feet at an elevation of 103.5 feet. An analysis of floods of reduced magnitude indicates that a flood magnitude of 30 per cent of the PMF ordinates would safely pass the spillway.

Results of the routing analysis and a summary of initial routing conditions are presented in Table 1.

TABLE 1: ESTIMATED PMF ROUTING SUMMARY

	Existing Conditions	Design Conditions
Starting Pool Elevation	101.0 ft.	96.0 ft.
Starting Pool Storage	1,870 acre-feet	1,330 acre-feet
Inflow Peak	6,635 cfs	6,635 cfs
Outflow Peak	1,925 cfs	635 cfs
Storm Volume	660 acre-feet	660 acre-feet
Maximum Storage	2,180 acre-feet	1,883 acre-feet
Maximum Pool Elevation	103.5 ft.	101.1 ft.

Key Elevations:
96.0 feet - Spillway Crest
101.0 feet - Top of Saddle Dam
101.0 feet - Top of road embankment
105.0 feet - Top of Chessman Main Dam

2.3.4.2 Original Design Conditions

Routing of the PMF under existing project conditions is controlled by the road embankment upstream of the spillway. A routing analysis was also performed which reflects original design conditions and neglects the effects of the road. For the original design routing, it was assumed the road embankment was removed and the spillway was clear of debris and flashboards to permit unobstructed flow. A routing under these conditions and beginning at a pool elevation of 96 feet shows the reservoir level will rise to elevation 101.1 feet, which is 0.1 feet above the saddle dam crest. The limited depth and duration of the saddle dam overtopping suggests the project might safely handle the PMF if restored to its original design condition.

2.4 GEOTECHNICAL EVALUATION

Information presented in this geotechnical evaluation is based on as-built drawings, observation of the structures by field inspection, and interviews with the present and past operators of the dam. No geotechnical data per se have been collected by investigation of the structures for this report, nor are any geotechnical data present in records available for the dam except for the as-built drawings.

2.4.1 Chessman Main Dam Embankment

The measured dimensions of Chessman Main Dam embankment recorded during the inspection conform well to the as-built drawings. The measured crest length is 440 feet and the measured height of the fill is 40 feet, which is as shown on the as-built plans. The as-builts show a crest width of 15 feet, whereas the measured crest width is 17 feet. The measured backslopes on the upstream and downstream faces of the dam conform to the as-built specifications of 1V on 2H with some minor variations discussed in the following sections.

2.4.1.1 Embankment Material

The as-built drawing calls out the main dam embankment as "selected material, rolled." As part of the inspection, a single hand-auger hole was bored into the downstream face of the dam at a point approximately 11 feet below the crest. The material recovered from the auger hole consisted of decomposed quartz monzonite sand to a depth of 42 inches where the auger was stopped by a rock. The auger resistance indicated penetration of several decomposed quartz monzonite rock fragments in place in the fill. The quartz monzonite sand is a silty sand (SM) whereas kaolinitic clay content in the decomposed rock fragments results in a clayey sand (SC).

The main source of the material used as fill in the embankment is not shown on any of the old plans, nor is it evident in the field. The photographs of the dam during construction were taken after the reservoir area was partially filled so the area adjacent to the upstream toe of the dam is not visible. However, the photographs show that the fill material was transported to the embankment by means of horse-drawn equipment. The limited capability of the horse-drawn earthmoving equipment to transport fill over long distances and the lack of any obvious borrow pit at the dam (excepting some small cuts on the left and right abutments) suggest that the primary source of borrow material for the embankment was within the reservoir area.

As previously discussed, local geology strongly indicates that the original natural ground within the reservoir area consists of sand and silty sand with the gravel fractions containing a considerable amount of quartz monzonite and granodiorite. Thus, the material observed in the shallow hand auger hole in the embankment may be representative of the bulk of the embankment.

2.4.1.2 Embankment Crest

The crest of the main dam is unpaved and consists of decomposed quartz monzonite silty sand (SM) that has been compacted by traffic throughout the years. Some irregularities in grade exist near the control valve house. Otherwise, spot elevations taken along the crest of the dam and visual examination of the crest alignment indicate that no differential movement or unusual settlement of the embankment mass has taken place.

2.4.1.3 Upstream Face

The upstream face of the main dam is constructed with a 1V on 2H backslope (as field checked with a Brunton compass) and is riprapped with a flagstone pavement. The flagstone consists of hand-placed quartz monzonite roughly 1 cubic foot in size and is bedded in the embankment without mortar. The quartz monzonite is coarsely crystalline, but remains in good condition above the normal full storage pool.

The flagstone at the elevation of the normal full storage pool (approximately 6.5 feet below the crest elevation) where wave action is the most severe and prolonged shows signs of deterioration. Many fragments of flagstone 3 to 4 inches in mean dimension have separated from the pavement in a 6- to 8-inch zone at this elevation. Flagstone observed to a depth of 2 to 3 feet below the water surface did not exhibit signs of this deterioration. The flagstone in the zone of deterioration is still performing as a satisfactory riprap, but eventually it will be desirable to replace the deteriorated stone pavement at this level.

A fan-shaped portion of the flagstone pavement on the upstream face south of the valve control house is devoid of fines between the flagstone. The remainder of the exposed pavement typically has fines filling the spaces between the flagstones. Positive evidence of downslope displacement of the flagstone pavement is not discernable. An elevation profile along the upper edge of the flagstone at the crest reveals an elevation loss of less than 0.1 feet along the area devoid of fines that may easily be within the magnitude of relief along the flagstone surface. The area devoid of fines also corresponds to an area of minor slope failure on the downstream face of the embankment.

The upper edge of the flagstone pavement progressively decreases in elevation from about 105.5 feet at Station 0+50 to 103.4 feet at Station 0+00. All stations are measured in feet from the right (north) abutment. Although the crest is graded from front to back to conform to the edge of the flagstones, the center and downstream portions of the crest do not reflect the loss in elevation. It is not known whether the feature reflects a downslope shift in the flagstone or a small displacement of a portion of embankment fill by slope failure or if it is simply an imperfection in the final configuration of the dam as constructed.

2.4.1.4 Downstream Face

The downstream face of the main dam is constructed with a 1V on 2H backslope. Careful inspection of the backslope shows that it is somewhat concave in profile due to the upper one-fourth being slightly steeper than the lower three-fourths. Surface material consists of silty sand and friable rock fragments derived from deteriorated quartz monzonite as previously described. It is moderately vegetated with grass, wild rose bushes, seedling pine, and two clumps of willow which are widely separated on the face. The vegetation is evidently mowed by a maintenance crew on a regular basis. Unusual features observed on the downstream face include a small slope failure, distinctive zoning of vegetation on the slope and an area of incipient accelerated erosion.

The slope failure is located approximately between stations 2+30 to 2+50 as shown on Plate 3. The failure is expressed as a small tension crack approximately 12 feet below the crest on the downstream face with less than one foot of vertical displacement (presently overgrown with heavy grass) below which is a somewhat irregular slope surface terminating in an incipient toe bulge at the toe of the dam. The failure appears to be a case of shallow soil creep which accelerated into a local shallow soil detachment and may be indicative of seepage. The soil creep is an explanation for the slightly concave shape of the backslope.

Vegetation on the downstream slope of the dam exhibits distinct zoning. The uppermost 7 feet of the embankment is vegetated almost exclusively by grasses and many areas of sandy appearing soil are exposed in the sparse cover. Below an average level of 7 vertical feet below the crest elevation, numerous wild rose bushes are present on the downstream face. Wild rose bushes are generally indicative of the presence of high soil moisture. In addition, a distinct increase in the growth of grasses on the downstream face is visible from about 10 to 12 vertical feet below the crest on down to the toe of the embankment. The increased development of grass growth corresponds to the lower limb of the concave downward backslope profile. The zoned vegetation may be indicative of the location of a wetting front or seepage front in the downstream face of the dam but may also be related to soil textures on the downstream face. The presence of the zoned vegetation indicates that the possibility of a phreatic water surface at a high elevation in the fill must be given serious consideration as a factor in embankment stability.

The area of incipient accelerated erosion is located adjacent to the control valve house and consists of the beginning of gully erosion down the face of the dam. Drainage on the crest of the dam is away from the downstream face (Photo 13). The erosion is probably the result of a snowdrift forming against the west side of the control valve house and melting over the downstream face in the spring.

2.4.2 Chessman Main Dam: Foundation Conditions, Seepage, and Drainage

Preconstruction investigations and plans do not include drill hole data or other subsurface information regarding foundation materials. However, engineering drawings labeled "Chessman Reservoir Dam, as used, 1907" (Plate 3) depict profiles for both the "natural surface of the ground" and the "bedrock". A scaled section through the embankment, outlet works, and core wall on this same drawing shows the core wall extending down to approximate bedrock elevation in the location of the valve chamber, suggesting that the core wall is founded on bedrock although the core wall foundation or base is not specifically called out in the profile view. Photographs 4 and 5 taken during construction of the dam show the concrete core wall extending along the entire length of the dam. The contact of the core wall with bedrock and the top elevation of the core wall could not be verified by this investigation.

2.4.2.1 Foundation

The above information indicates that the concrete core wall of the main dam may be founded on bedrock and that the embankment fill and the outlet conduit are founded in alluvium. Examination of the site geology during the safety inspection revealed geologic relationships that indicate the foundation bedrock consists of quartz monzonite. Time limitations and the cover of residuum and forest precluded detailed investigation and mapping of faults or other secondary structures in the immediate vicinity of the dam. However, geologic mapping presented in U.S. Geological Survey Professional Paper 428 (Reference 1) at a scale of 1:48,000 does not show faults or structural trends crossing the dam site or reservoir area (Plate 5).

Test pit or drill logs and other sample data of the alluvium are not available. The nature of the source area for the alluvium indicates it consists of sand, clay and gravel mixtures. The maximum thickness of alluvium shown on the as-built drawing is approximately 30 feet.

The inspection revealed no evidence which would indicate that significant differential settlement has occured in the foundation material.

Inspection of the toe of the downstream face and areas downstream did not reveal seeps or wet areas, concentrations of phreatophytic vegetation, washed areas, or other evidence of significant seepage through the alluvial foundation materials. The absence of surface seepage and the generally excellent drainage characteristics of the alluvial material suggest that seepage conditions in the alluvial foundation do not offer any stability problems.

Examination of surface exposures of the bedrock foundation materials shows essentially no intrinsic permeability in the coarsely crystalline quartz monzonite. Joints and fractures are small and widely spaced in outcrops near the dam. Moreover, hydrothermal alteration of the quartz monzonite exposed on the right abutment has altered feldspar minerals to clay, further reducing the potential of secondary permeability in joints or fractures.

2.4.2.1.1 Right Abutment The main dam right abutment consists of quartz monzonite overlain by quartz latite. The core wall and dam embankment abut quartz monzonite and the quartz latite contact is located above all but a few feet of embankment fill on the downstream side of the core wall. A northwest by southeast trending chalcedony vein is exposed in this abutment in a low cut along the reservoir bank approximately 30 feet north of the embankment fill where it is marked by red iron stains. The quartz monzonite adjacent to this vein is hydrothermally altered in situ to a friable clayey sand soil exhibiting relict igneous structure. The altered bedrock is easily excavated with a shovel. The altered zone is at least 25 feet wide and of unknown depth (Plate 5).

There is no evidence of seepage through the right abutment, possibly in part due to the clayey nature of the hydrothermally altered quartz monzonite. There is no evidence of movement or deformation due to piping of material, settlement, shear failure, or plastic deformation; however, the altered bedrock appears to be a soft seam transecting the abutment. Further investigation of this feature is not warranted in view of the long history of stability and lack of any evidence of seepage or deformation. Moreover, the vertical dip of the zone and its strike alignment suggest that it supports little, if any, of the compressive load of the dam.

2.4.2.1.2 Left Abutment The main dam left abutment consists entirely of quartz monzonite and is an extension of the bedrock foundation under the dam. Solid bedrock is not exposed on the abutment which is covered by 3 to 5 feet or more of colluvial slope deposits. Bedrock exposed in shallow cuts on the downstream side of the abutment consists of highly weathered, friable, decomposed quartz monzonite in situ; however, the rock has not been hydrothermally altered. No observation of joint development or spacing could be made on this abutment. There is no evidence of seepage or slope instability in either the bedrock or the colluvium.

2.4.2.2 Main Dam Seepage Conditions

At the time of the field safety inspection on August 9, 1978, the water surface in the reservoir was 7 vertical feet below the crest of the dam (crest elevation = 105.0 and water surface elevation = 98.0). All elevations are based on an arbitrary elevation of 105.50 feet assigned to the top of an iron pin located midway between the valve chamber house and the right abutment. The observed level of storage corresponds to a storage level of about 28.0 feet on an arbitrary staff gage used by the operator, and is 1 foot lower than the defined normal pool elevation of 99 feet (29 feet gage). The operator said he maintains the pool somewhat lower than normal pool elevation because previous operators reported observing seepage at the toe of the dam when the pool exceeded 99 feet. The toe of the dam and the area downstream from the dam for several hundred feet in the natural alluvium were carefully examined for signs of a wetting front or seepage, but no areas of unusual soil moisture content, free seepage or unusual phreatophyte development were observed.

A 10-inch-diameter cast iron (CI) pipe was observed protruding from each bank of the outlet channel about 100 feet downstream of the outlet. One pipe is shown in Photo 16. These pipes were assumed to be drains and were discharging clear water at an estimated 5 to 10 gallons per minute. Another 10-inch CI pipe protrudes from the dam embankment directly above the outlet tunnel (shown in Photo 15), but was not discharging any water. No information concerning these pipes was found and they are not indicated on the as-built drawings.

Further inspection for seepage was conducted within the control valve chamber where seepage and a marked increase in moisture content on the concrete chamber walls were observed at a depth of 11.75 feet (elevation = 93.25 feet) below the crest of the dam on down to the bottom of the valve chamber. The seepage observed in the valve chamber was confined to the reservoir side of the chamber and consisted of free water seeping (but not flowing) on the concrete surface in several locations. Assuming that 93.25 feet is the upper elevation of the phreatic line through the embankment against the concrete core wall, a seepage gradient of 4.75 feet vertical to 16 feet horizontal exists and is not possible unless considerable seepage is passing through or under the concrete core wall. There are no piezometers or observation wells installed in the embankment or foundation downstream from the concrete core. Therefore, groundwater levels and pore pressures in this area are unknown.

2.4.3 Chessman Main Dam Embankment Stability

Inspection guidelines recommend a seepage and stability analysis be on record for dams such as Chessman and that the computed safety factor for steady seepage conditions be not less than 1.5. A review of all existing plans and documents pertaining to Chessman Dam reveals that a design seepage or stability analysis is not available for either embankment. Discussion of embankment stability is therefore limited to observed conditions.

The August 9, 1978 field inspection revealed the following circumstances that may be indicative of potentially unstable embankment conditions:

a. Evidence suggesting that soil creep and small failures may have occurred in the embankment in the past.

b. Evidence of zoned vegetation on the downstream slope which may be indicative of seepage-related soil moisture and a high phreatic surface in the embankment.

c. Evidence of significant amounts of seepage through the embankment and over, under, around or through the core wall as evidenced by an apparent seepage gradient between the reservoir and core wall at the valve chamber.

As described in Section 2.4.1.3 of this report, the upper edge of the upstream flagstone pavement between stations 0+00 and 0+50 is misaligned downward from the remainder of the dam crest. The upstream flagstone pavement near the valve control house similarly exhibits evidence suggestive of possible downslope displacement. The downstream face of the dam has what appears to be a shallow soil detachment failure between stations 2+30 and 2+50 opposite the area of possibly displaced flagstone on the upstream face. A possibility exists that the apparently shallow soil detachment may be a deeper recumbant shear failure.

The interpretation of seepage observed in the concrete control valve chamber (Section 2.4.2.2) indicates that seepage may be flowing from the upstream face of the dam to the downstream face of the concrete core wall. If the seepage interpretation is correct, the seepage must pass over, around, through or under the concrete core wall.

Information regarding the 1935 earthquake's effects on the structural integrity of the concrete core wall does not exist. Moreover, the actual as-built elevation of the top of the core wall is not factually documented, except in photographs showing the relationship between the top of the core wall and the valve chamber top on which the control valve house sits. The presence of zoned vegetation on the downstream face suggests that seepage flowing over or through the concrete core wall may be recharging a capillary wetting front that influences vegetation growth on the dam face and that the vegetation distribution and density may correlate with the location of the seepage. The location of soil creep and an associated slope failure (Section 2.4.1.4) on the downstream face may also relate directly to location of a phreatic seepage surface in the embankment and corresponds well with the distribution of vegetation zones.

If the foregoing evidence is truely indicative of the location of the phreatic seepage surface in the embankment, potentially dangerous pore pressures in the embankment may exist. In any case, sufficient information is not presently available to permit evaluation of embankment stability.

2.4.4 Chessman Saddle Dam: Embankment, Foundation, Seepage

2.4.4.1 Embankment

Chessman Saddle Dam is an earthfill structure, separate from the main dam. The actual dimensions of the saddle dam conform well to the as-built drawings. The crest length is 105 feet and the height is 17 feet measured from the toe of the downstream face. The upstream face slope is approximately IV on 1.5H, the slope of the downstream face is approximately IV on 2.5 H and the breadth of the crest is 10 feet. The embankment is reported to consist of rolled selected earth upstream of the core wall and rolled earth downstream of the core wall.

A concrete core wall extends the entire length of the embankment and is located along the upstream edge of the embankment crest. It appears from the as-built drawings (Plate 3) that the core wall is founded on bedrock. The wall extends approximately 4 inches above the top of the embankment to extablish the dam crest elevation of 101 feet (Photo 35). The observed horizontal alignment of the core wall does not indicate any tilting or lateral wall displacement and no transverse cracking was observed in the exposed portion of the wall.

The full 10-foot breadth of the embankment crest is paved with a concrete slab (Photo 36) which abuts the core wall. The slab is cracked with small trees growing through the concrete and in the joint between the slab and the core wall. Small bushes are growing on the upstream face of the embankment and Aspen trees 6 to 10 inches in diameter are growing on the downstream face. No riprap or embankment protection exists on the embankment slopes.

2.4.4.2 Foundation

Inspection of the abutment and foundation geology at the saddle dam indicates that the bedrock foundation material consists of relatively fresh quartz monzonite. The rock on the outcrop of the abutments require one to two hammer blows to break a 4 inch piece with a rock pick. The rock is cut by cleavage joints with a density of 2 to 3 per foot. The joints have sharp, distinct development but close to fine cracks below the weathered zone.

2.4.4.3 Spillway

The as-built drawings show the spillway to be founded on rolled backfill and natural ground. The spillway floor and training walls are paved with a concrete finish surface presumably over the original riprap indicated on the as-built drawings. Observation of the soil beneath the spillway chute was not possible during the inspection. However, small cracks and some spalling have exposed the rock surface below. Wall and floor alignments appear to be true with no significant evidence of settlement or movement. Hammer soundings indicate that the concrete surfacing is sound and no voids were detected. The cracking and spalling are not considered excessive or serious. Much of the concrete in the chute is covered with sod and some bushes are growing through the concrete. No riprap or embankment protection exists below the spillway at the toe of the embankment and this area would be subject to erosion during high flows affecting embankment stability.

2.4.4.4 Seepage

Because of the road berm, which crosses the approach channel, ponding behind the saddle dam does not reach the reservoir elevation. The pond between the saddle dam and the road berm was 2 feet lower (elevation 96 feet) than the reservoir at the time of the inspection which would indicate a seepage flow exists across the road berm. It would be expected that a corresponding outflow would exist from the pond. However, no water was being spilled and no seepage was observed in the saddle dam foundation abutments. The conditions observed at the time of the inspection were inadequate to support conclusions regarding seepage and stability conditions.

2.4.5 Peripheral Embankments

In addition to the main dam embankment, Chessman Reservoir is confined by two additional road berms. The Beaver Creek road berm crosses the entire extent of the upstream end of the reservoir and was evidently constructed for the purpose of confining sediment and floatables to an area outside of the reservoir. This berm is approximately 1000 feet long, 16 to 20 feet in width, and is at an average elevation of 103.0 feet. It is constructed of uncompacted sandy silt (SM) and clay (CL) with a riprap face on the main reservoir side. Two 18-inch diameter CMP culverts and one 10-inch diameter pipe provide a connection across the berm. The crest of the berm is rutted.

A second berm is a road embankment across the northeast edge of the reservoir. This berm, approximately 350 feet long, separates the reservoir from the spillway and is constructed of native material similar to the other road berm. The crest of the berm is 12 to 15 feet wide and at its lowest point (elevation 101 feet) is approximately 4 feet below the crest of Chessman Dam and approximately 5 feet higher than the spillway crest. The berm is essentially at the same elevation as the as-built elevation of the core wall in the main dam.

If culverts were originally placed through this fill, they were not evident during the inspection and a differential of 2.1 feet in water surface elevations across the berm indicates that only limited hydraulic communication exists across the berm. An apparent hydraulic gradient across the road berm indicates seepage flow exists from the reservoir to the pond behind the spillway. If the condition, as observed, is a steady state condition then it suggests that there is a corresponding outflow from the pond, presumably through the saddle dam embankment or foundation. The absence of observed seepage downstream from the saddle dam embankment and abutments indicates no significant seepage is taking place when the pond between the spillway and the road berm is at elevation 96 feet. The absence of observed outflow from the pond supports the conclusion that minimal seepage flow is occuring across the road berm.

2.5. PROJECT OPERATIONS AND MAINTENANCE

2.5.1 Chessman Main Dam And Chessman Saddle Dam

The City of Helena, Department of Public Utilities has operational and maintenance responsibility for Chessman Dam. From observations at the time of the inspection it appears that the main dam is well maintained although there is no formal maintenance plan. Vegetation on the downstream face of the dam consists mainly of grasses, small bushes, and seedling trees and is apparently mowed periodically to prevent deterioration of the face due to vegetative overgrowth. While the main dam apparently receives routine maintenance, the saddle dam has apparently not received any maintenance work for several years. This is possibly reflective of the fact there is no formal maintenance plan for the saddle dam and that the dam is not in use.

The dams are visited at least weekly by the operator, who is an employee of the Helena Department of Public Utilities, and more frequently during periods when operational needs require the operators presence. Formal inspections of the dam and appurtenances are conducted annually by representatives of the Helena National Forest, since the dam and reservoir are located within the Forest and are operated under a special use permit.

2.5.2 Reservoir Regulating Plan

As a water supply storage reservoir the Chessman Reservoir is used to supplement the water supply from Helena's Ten Mile Creek system when water right requirements from Ten Mile Creek cannot be met. Chessman Reservoir is filled to normal operating pool (elevation 99 feet) as early as possible in the spring and maintained at normal pool until releases are necessary to augment the Ten Mile Creek flows. In some years, releases from Chessman Reservoir are not required until early fall.

2.5.3 Warning System

No formalized plan exists for warning downstream inhabitants of emergency conditions at the dam or for forecasting the magnitude of potentially high inflows to the reservoir.

2.6 POTENTIAL FOR DOWNSTREAM DAMAGE

2.6.1 Chessman Main Dam and Beaver Creek Drainage

The evaluation of downstream hazard potential in this report is based on visual observation and engineering judgement. A detailed hydraulic routing of a dam breach hydrograph is required to accurately assess the potential for downstream damage and effects of dam failure.

The Phase I inspection analysis indicates that under PMF conditions the Chessman Main Dam will not be overtopped. However, should the embankment fail for any reason, a flood wave would be generated down the narrow canyon, the effects of which would greatly exceed the effects of a prefailure flow condition in the drainage. Approximately 20 to 25 residences in Rimini would be in the direct path of a flood wave, as would an additional 10 residences spotted along Ten Mile Creek below Rimini.

2.6.2 Chessman Saddle Dam and Buffalo Creek Drainage

The project spillway is located in Chessman Saddle Dam and discharges to the Buffalo Creek drainage. Potential for downstream damage and loss of life resulting from dam failure was evaluated under two operating conditions as is discussed in Section 2.3.4 of this report. Under existing reservoir operating conditions the Saddle Dam would over-top by 2.5 feet and presumably fail. Failure would cause a flood surge downstream and a significant increase in the potential for loss of life and property damage over a prefailure condition.

CHAPTER 3

FINDINGS AND RECOMMENDATIONS

3.1 FINDINGS

Chessman Reservoir is formed by two separate earthfill structures; Chessman Main Dam which is 40 feet high and contains the regulating outlet works, and Chessman Saddle Dam which is 17 feet high and contains the project spillway. At normal operating pool, project datum elevation 99 feet, the project impounds 1,630 acre-feet of water. At the saddle dam crest, elevation 101 feet which is the maximum elevation of spillway control, the project impounds 1,870 acre-feet.

Inspection criteria classifies these dams as intermediate in size and because of possible downstream effects of failure, the downstream hazard potential is high (Category 1) in both the Beaver Creek and Buffalo Creek drainages. Inspection criteria recommends such projects to safely handle a full PMF. The PMF determined for this preliminary investigation resulted from a 6-hour summer storm having a total precipitation of 10-inches. The resulting flood would have a peak inflow of 6,635 cfs and a volume of 660 acre-feet. Routing of the PMF through the reservoir is controlled by a roadfill, elevation 101 feet, which has been constructed across the spillway approach channel. This roadfill is critical to the safety of the project in that it prevents discharge through the spillway until the pool has reached an elevation of 101 feet which is the elevation of the saddle dam crest. Under this existing operating condition the spillway is inadequate due to the loss of routing capabilities. With the presence of the roadfill, the saddle dam will be overtopped and presumably failed by a flood derived by taking 30 percent of the full PMF ordinates.

A second routing analysis of the PMF was performed which addressed the effect of removing the roadfill and restoring the saddle dam and spillway to original design conditions. With a starting pool elevation of 96 feet at the spillway crest, and with all flashboards removed, the PMF would overtop the Saddle Dam by 0.1 foot which suggests the project might safely handle the PMF if restored to original design conditions.

Even under present conditions a PMF will not cause Chessman Main Dam to be overtopped. Conditions were, however, identified at the main dam which indicate seepage an embankment stability are of major concern although sufficient information is not presently available for an evaluation of embankment stability.

3.2 RECOMMENDATIONS

The following recommendations need to be acted on without delay.

3.2.1 Immediately develop and implement an emergency downstream warning plan for the areas below both dams.

3.2.2 Draw the reservoir down to elevation 96 feet (spillway crest) and remove enough of the road berm which crosses the spillway bay to insure control is always maintained at the spillway.

3.2.3 Clean all debris and dirt and remove all stoplogs from the spillway chute to insure full capacity is available.

3.2.4 Perform necessary repairs to spillway chute and provide adequate protection downstream from spillway chute exit, as required, to insure stability during spillway operation.

3.2.5 Remove all trees, brush and stumps from both embankments and toe areas; backfill and compact the disturbed areas.

3.2.6 Place adequate and/or repair existing protection on the upstream embankments faces of both dams.

3.2.7 Repair riprap in the main dam outlet exit channel.

3.2.8 Inspect the outlet conduit and repair if damaged.

3.2.9 Apply corrective measures to prevent development of gully erosion on the downstream face of the main embankment adjacent to the valve house.

The above recommendations will not make the project conform to inspection guidelines but will reduce the risk of loss of lives and property while the following recommended actions are being taken.

3.2.10 Conduct more detailed hydrologic and hydraulic routing studies below both dams to better determine the downstream hazards and required spillway capacity and modify the project as indicated.

3.2.11 Conduct and place on file, stability and seepage analyses of the dam embankment. These analyses should be performed by a qualified geotechnical engineer and be based on: seismic and static loading conditions; insitu strength properties of the embankment, foundation, and abutment materials; and phreatic surface conditions. Determine the seismic loads based on a seismic assessment of the area; and also evaluate the project's response to the seismic event. Establish the material strength properties by drilling and sampling with laboratory testing as appropriate; and obtain the phreatic surface by installing and monitoring piezometers.

3.2.12 Prepare and implement a formal operation and maintenance plan including as a minimum:

a) Procedures for forecasting and monitoring flood conditions;

b) Emergency downstream warning procedures;

c) Routine inspection and maintenance plan by the owner;

d) Periodic inspection by qualified engineers or other technical experts at least once every five years.

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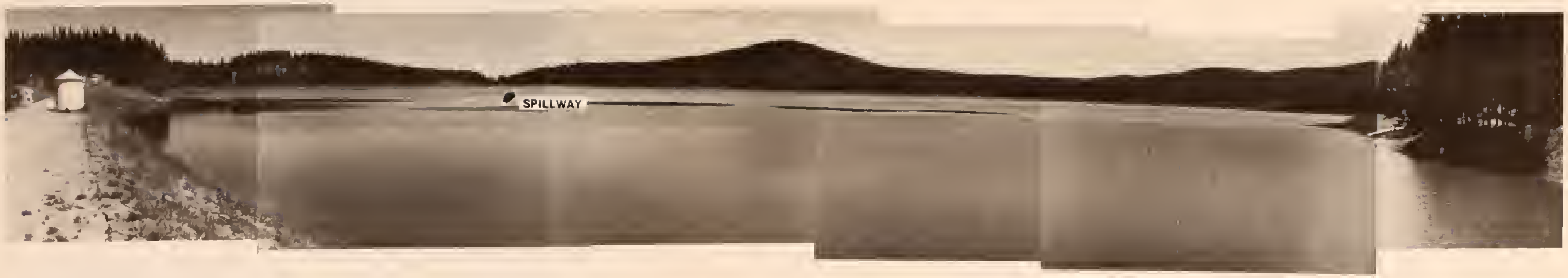


PHOTO 1 CHESSMAN RESERVOIR FROM MAIN EMBANKMENT
NOTE SPILLWAY LOCATION

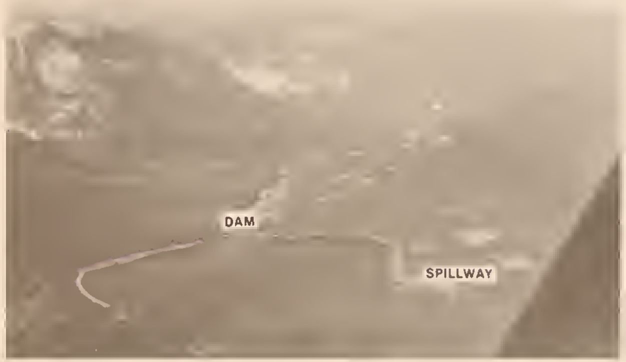


PHOTO 1A CHESSMAN RESERVOIR — AERIAL VIEW

CHESSMAN MAIN DAM

INSPECTION PHOTOGRAPHS TAKEN AUGUST 9, 1978

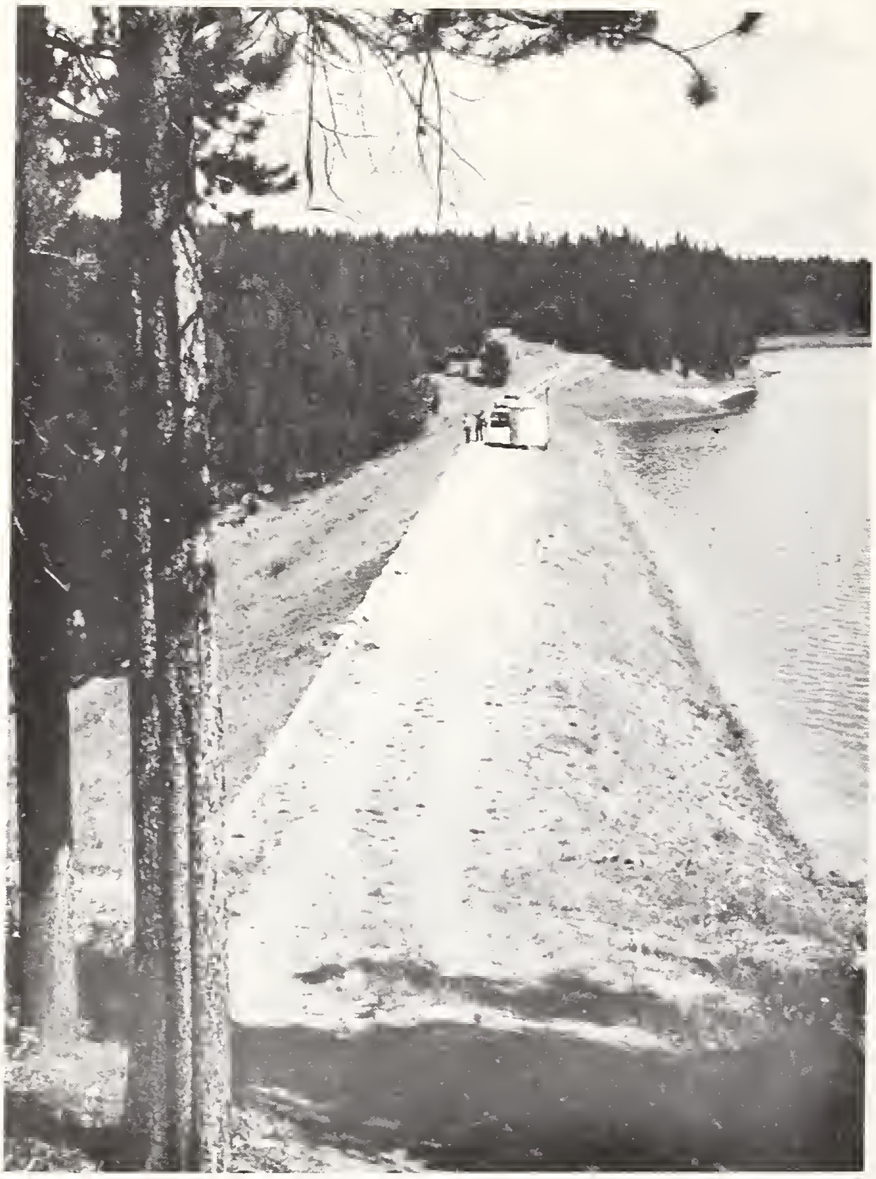


PHOTO 2 MAIN DAM FROM SOUTH ABUTMENT

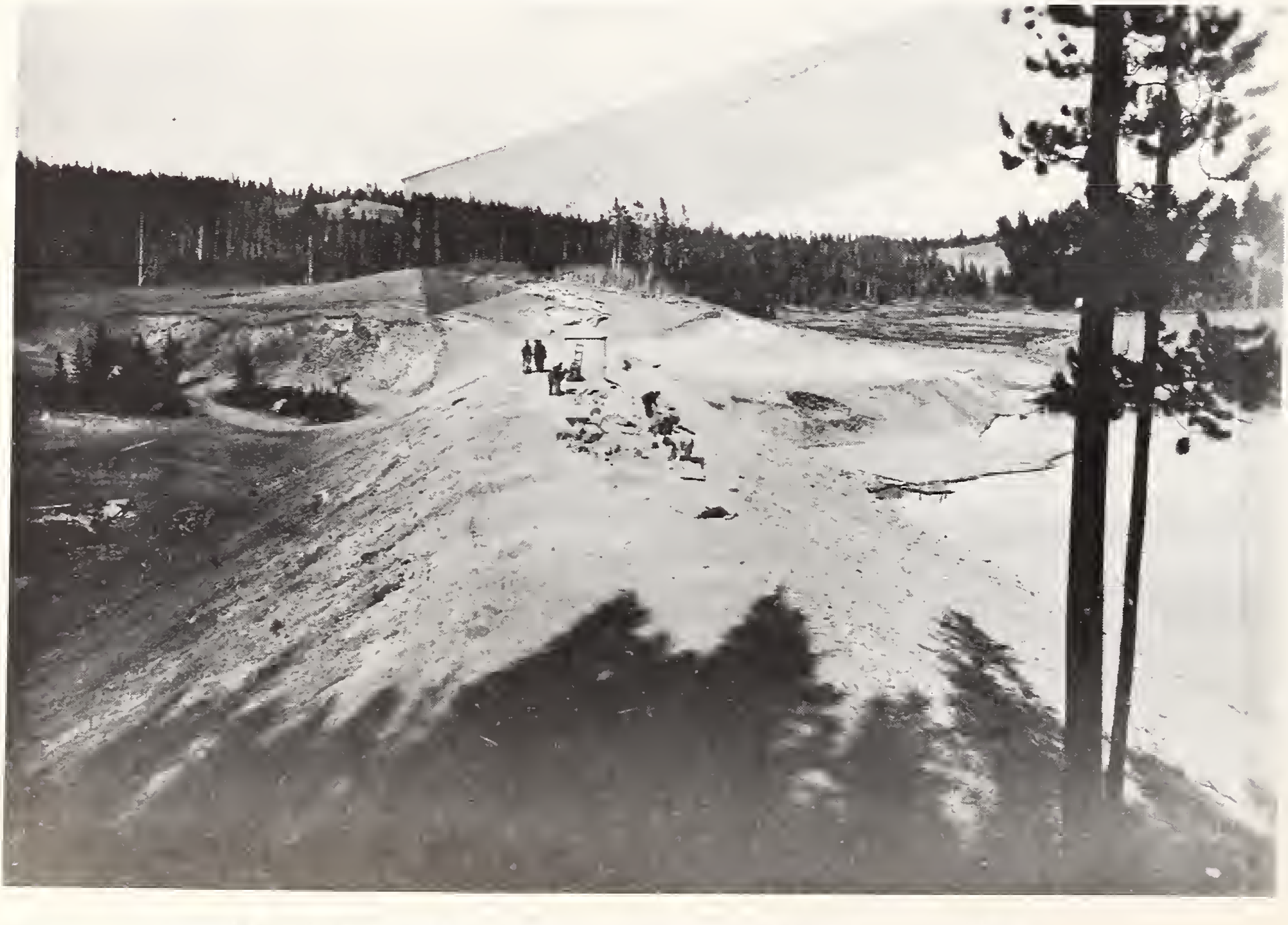


PHOTO 3 CHESSMAN DAM DURING CONSTRUCTION FROM SOUTH ABUTMENT
PHOTO COURTESY OF CITY OF HELENA

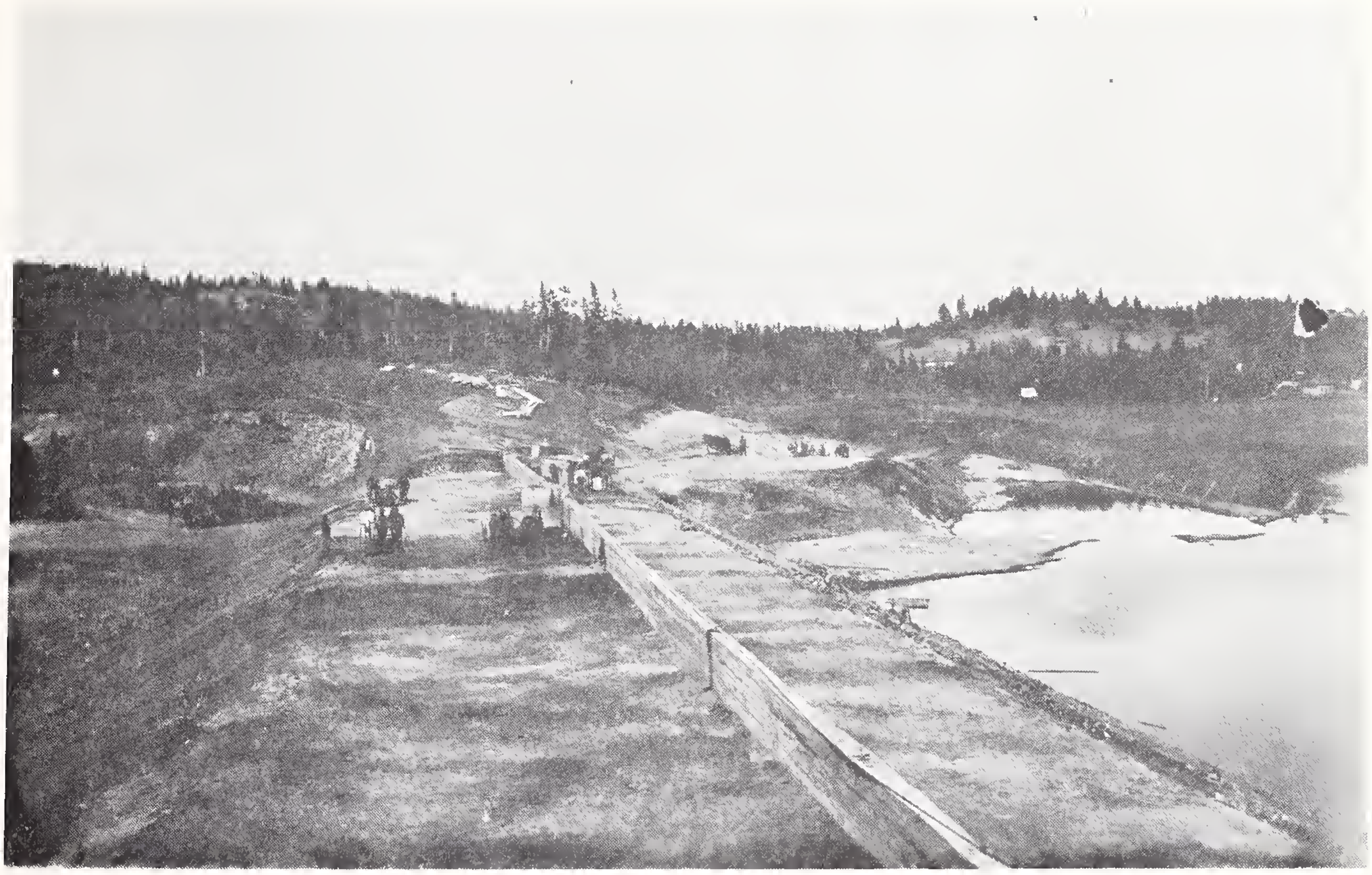


PHOTO 4 CHESSMAN DAM FROM SOUTH ABUTMENT DURING CONSTRUCTION, NOTE CONCRETE CORE WALL AND VALVE CHAMBER
PHOTO COURTESY OF CITY OF HELENA

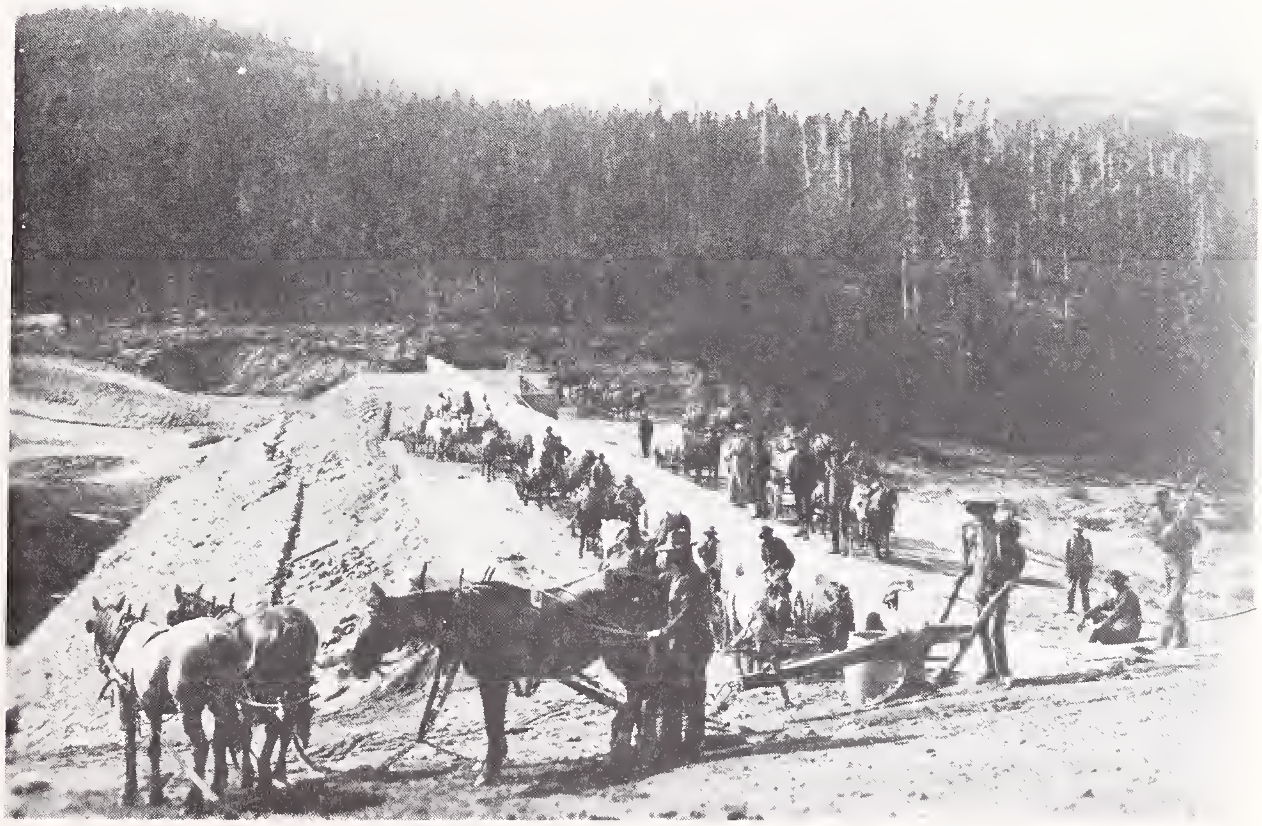


PHOTO 5 CHESSMAN DAM FROM NORTH ABUTMENT DURING CONSTRUCTION, NOTE HAND LAID RIP RAP
PHOTO COURTESY OF CITY OF HELENA

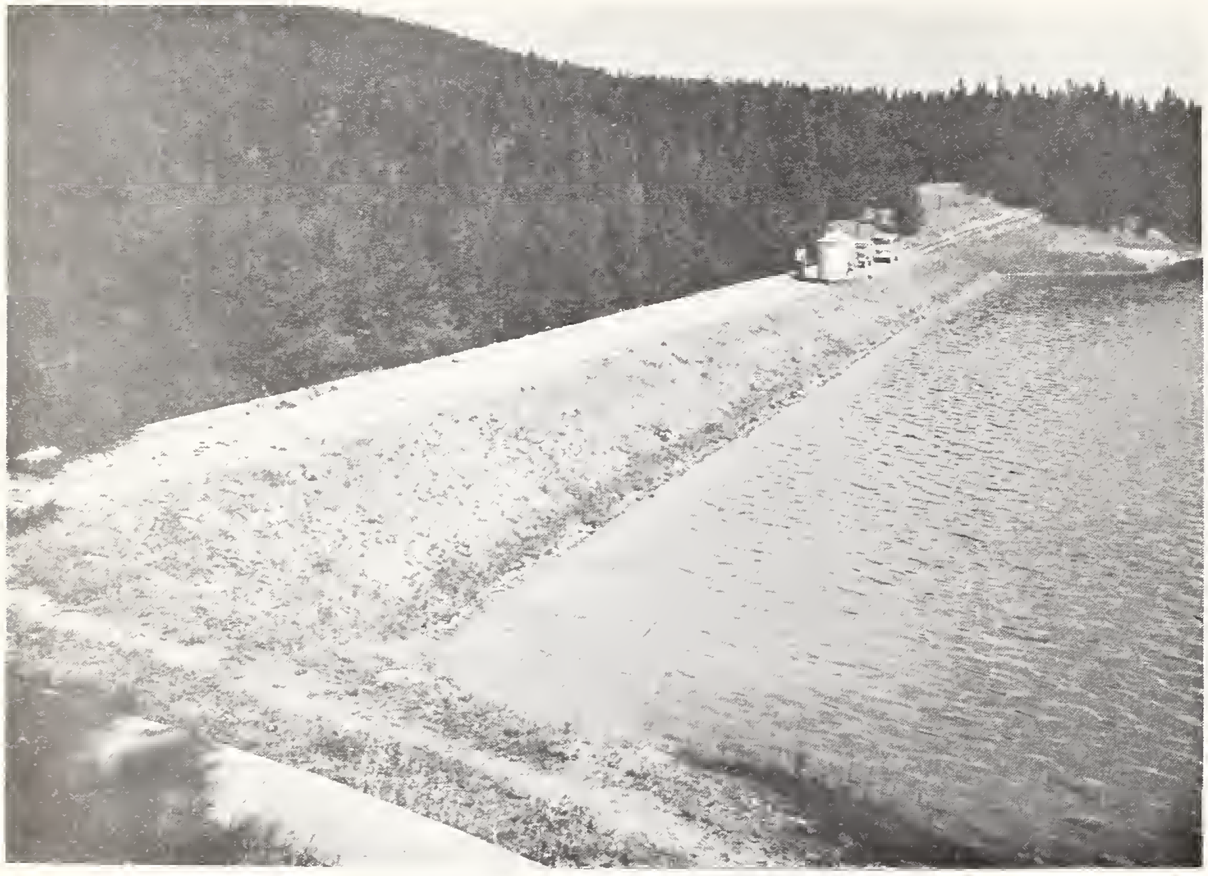


PHOTO 6 UPSTREAM FACE OF MAIN DAM

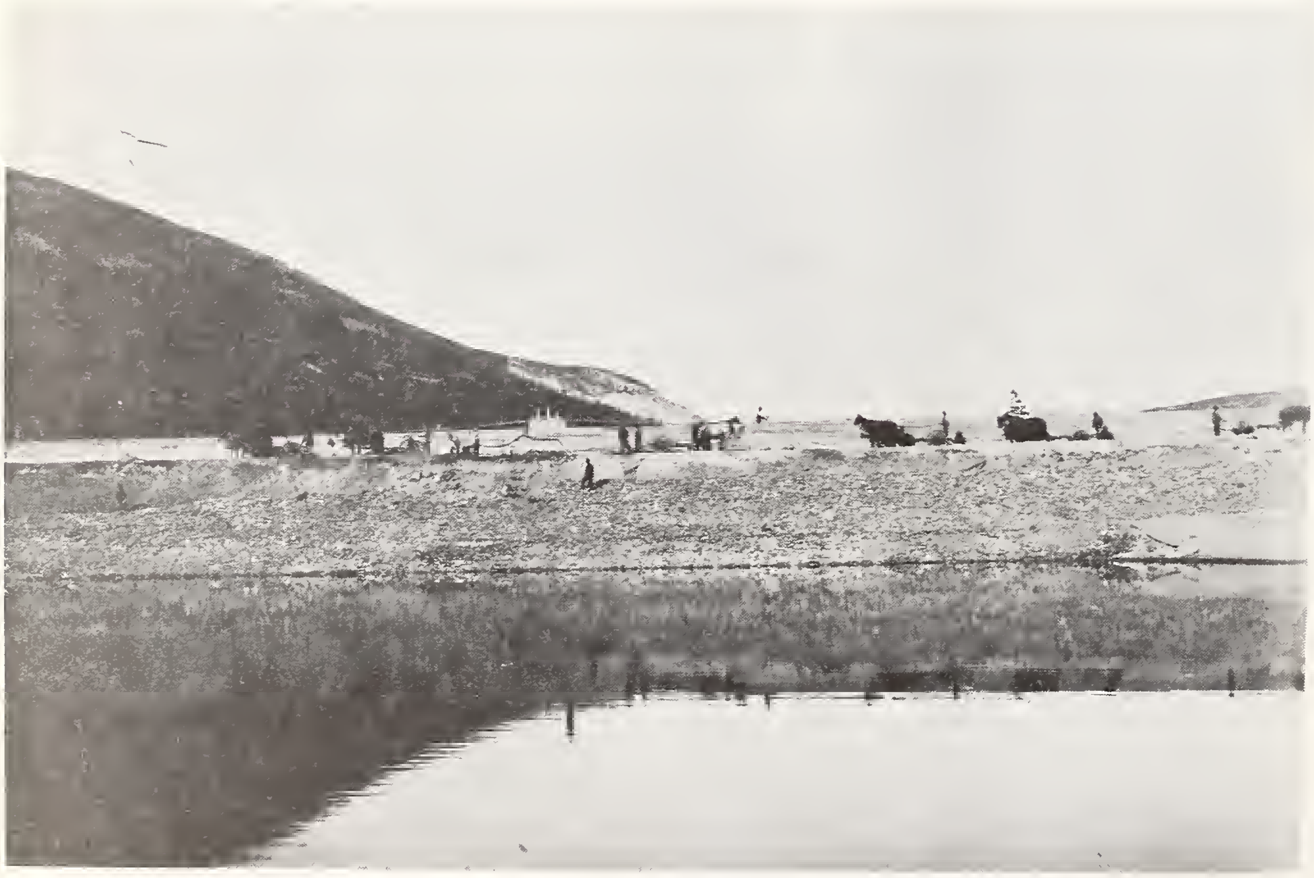


PHOTO 7 UPSTREAM FACE OF CHESSMAN DAM DURING CONSTRUCTION
PHOTO COURTESY OF CITY OF HELENA

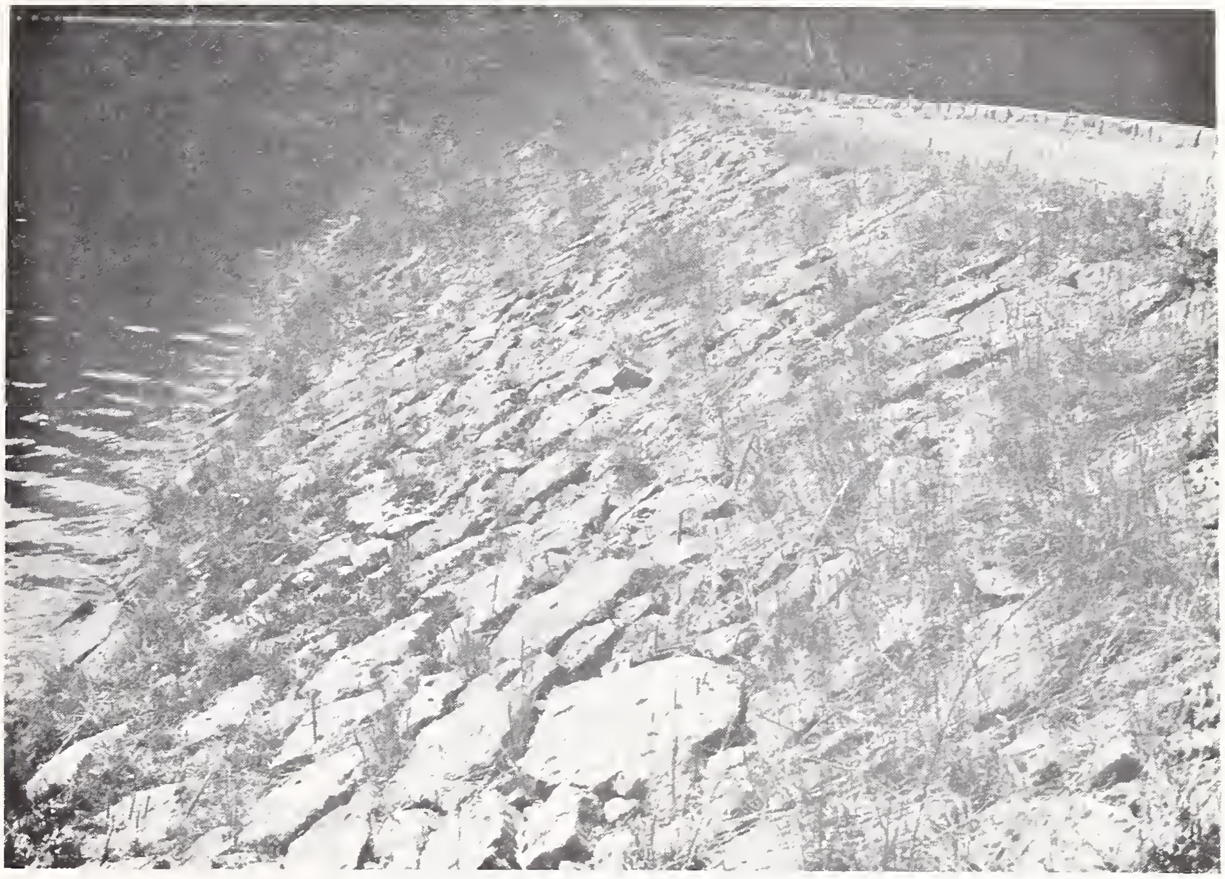


PHOTO 8 FLAGSTONE RIP RAP SOUTH OF VALVE CONTROL HOUSE

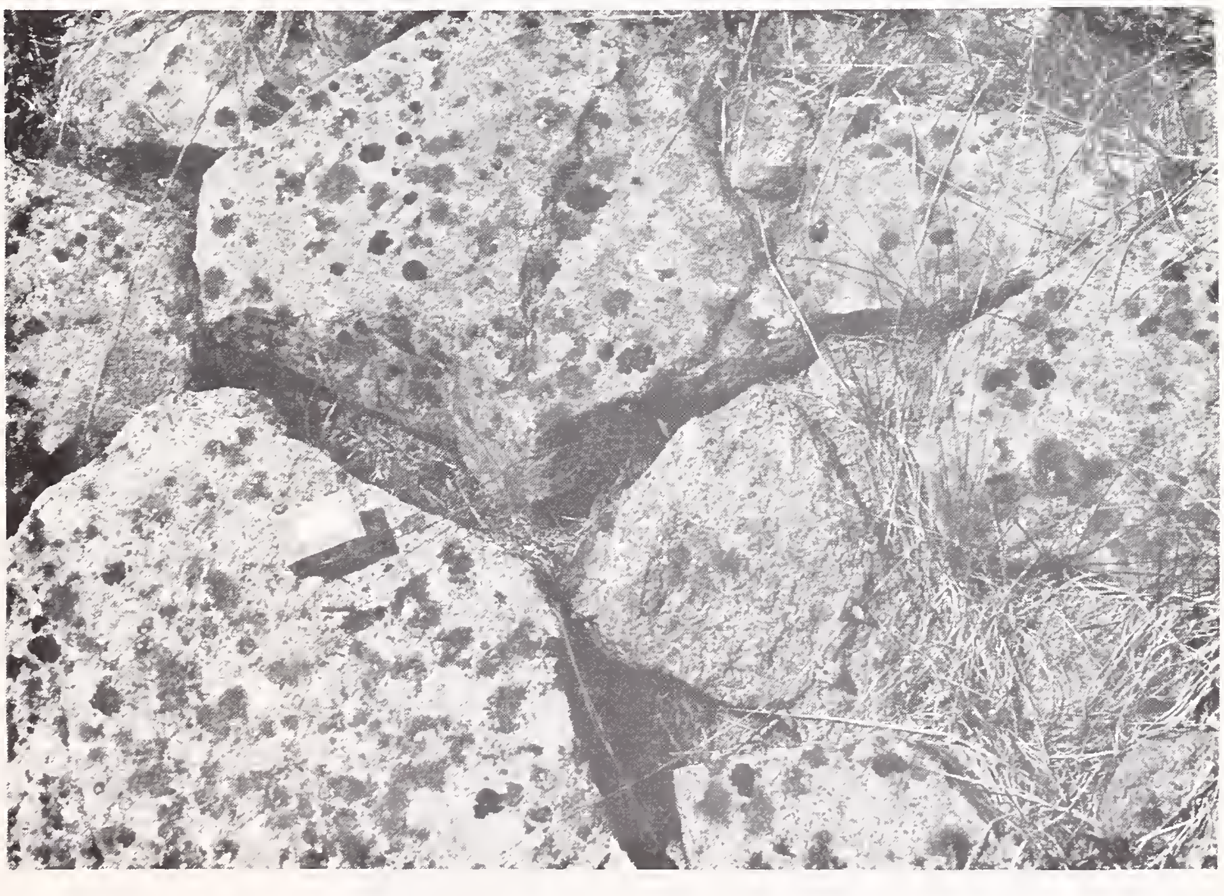


PHOTO 9 CLOSE UP OF RIP RAP. NOTE ABSENCE OF FILLER MATERIAL

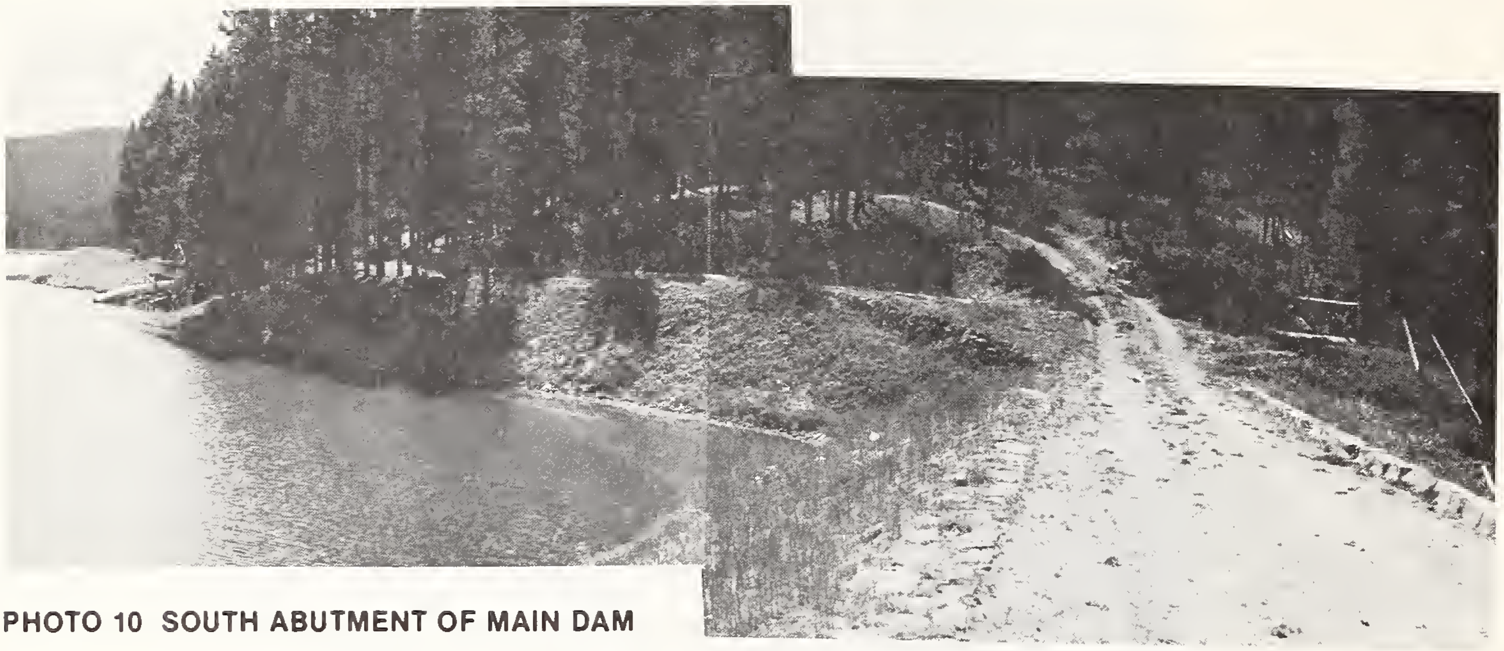


PHOTO 10 SOUTH ABUTMENT OF MAIN DAM

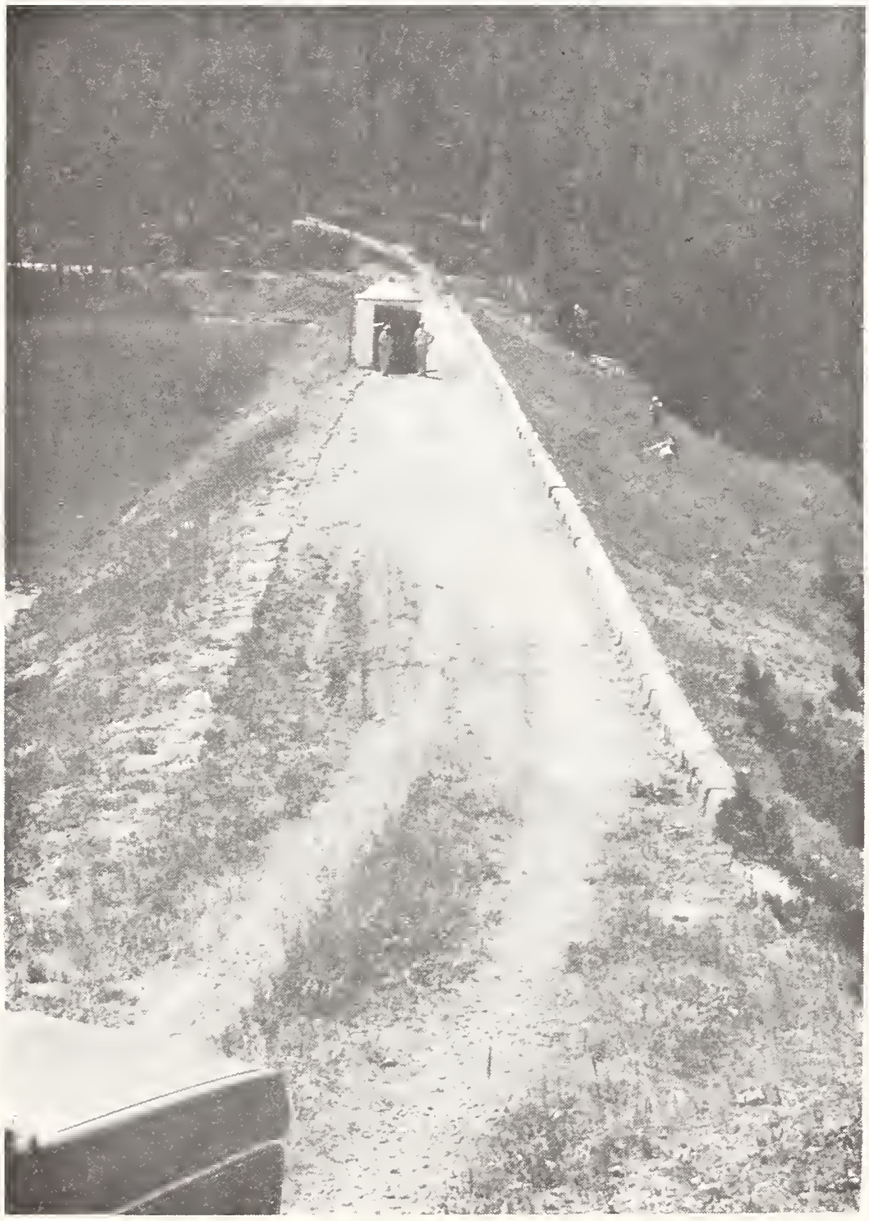


PHOTO 11 MAIN DAM FROM NORTH ABUTMENT

PHOTO 12 DOWNSTREAM FACE OF MAIN DAM LOOKING TOWARD SOUTH ABUTMENT

PHOTO 13 EROSION OF DOWNSTREAM FACE OF DAM AT VALVE CONTROL HOUSE

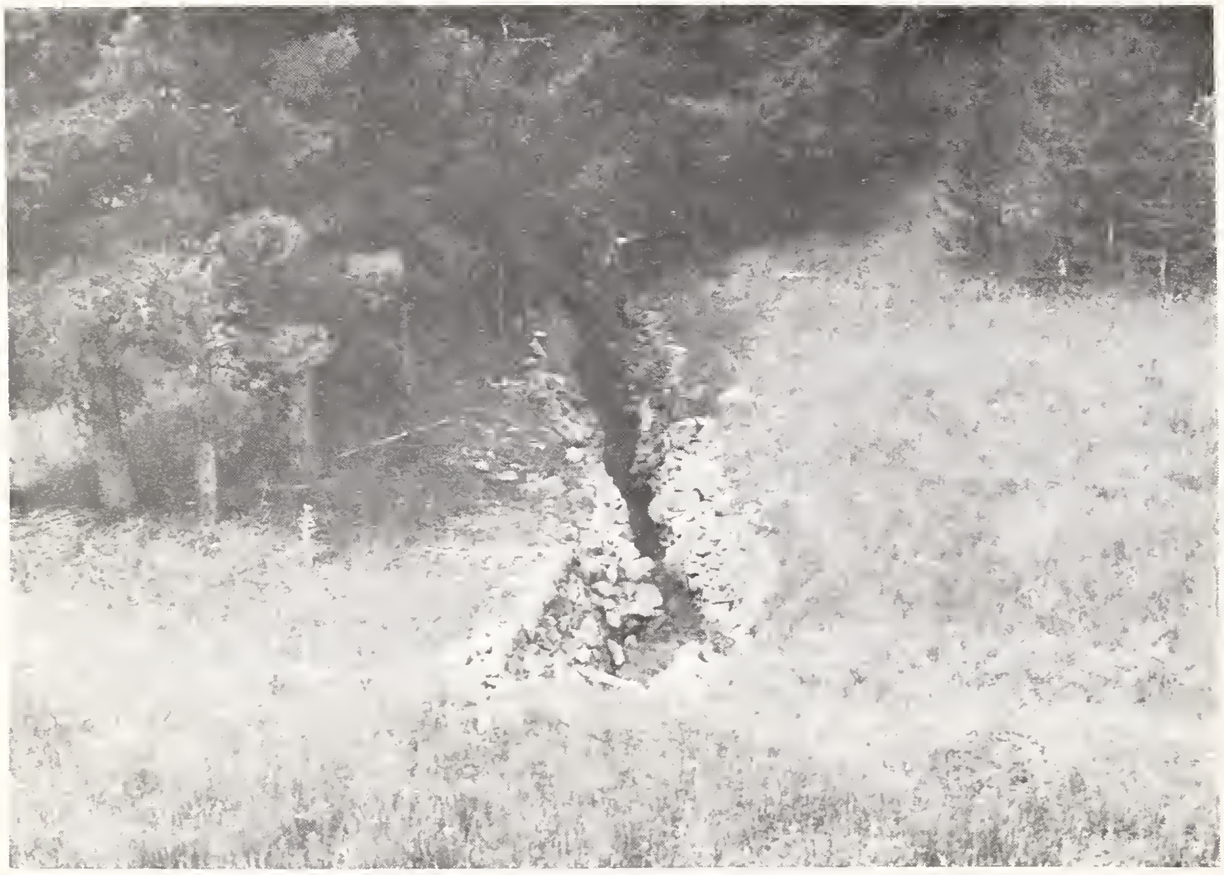


PHOTO 14 OUTLET DISCHARGE CHANNEL FROM TOE OF MAIN DAM

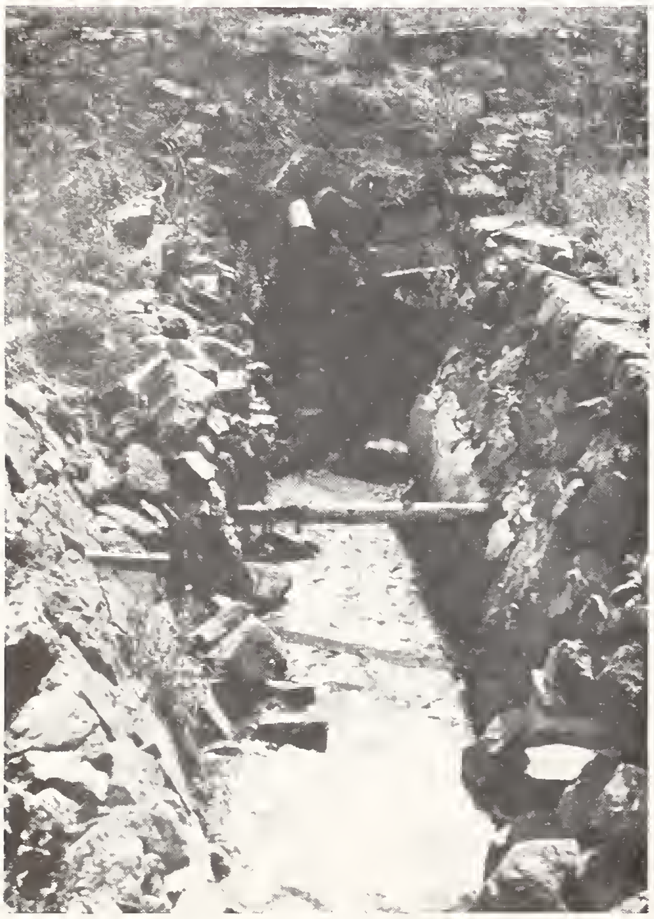


PHOTO 15 OUTLET TUNNEL AT TOE OF DAM

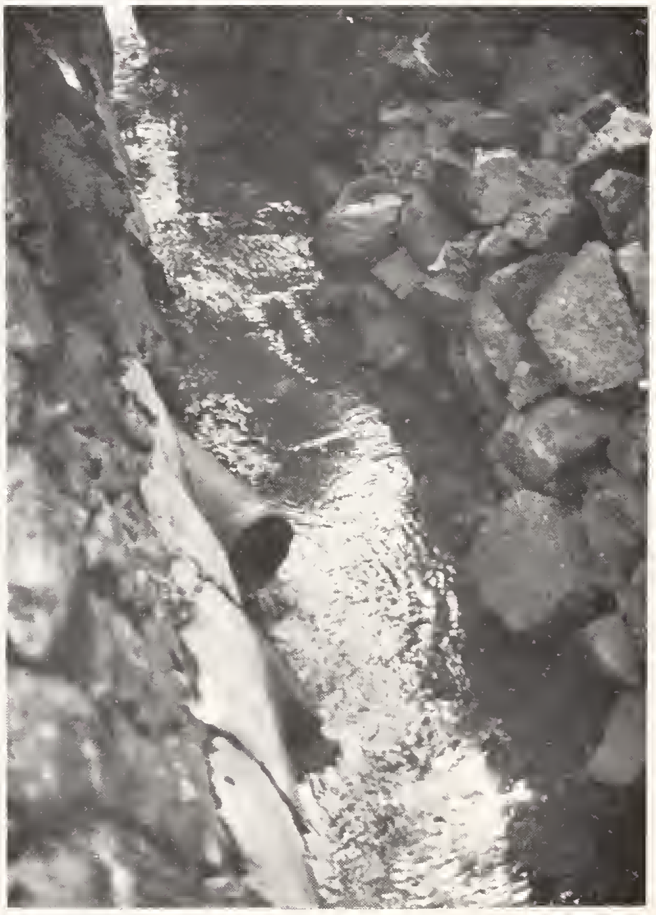


PHOTO 16 DRAIN DISCHARGING TO OUTLET CHANNEL

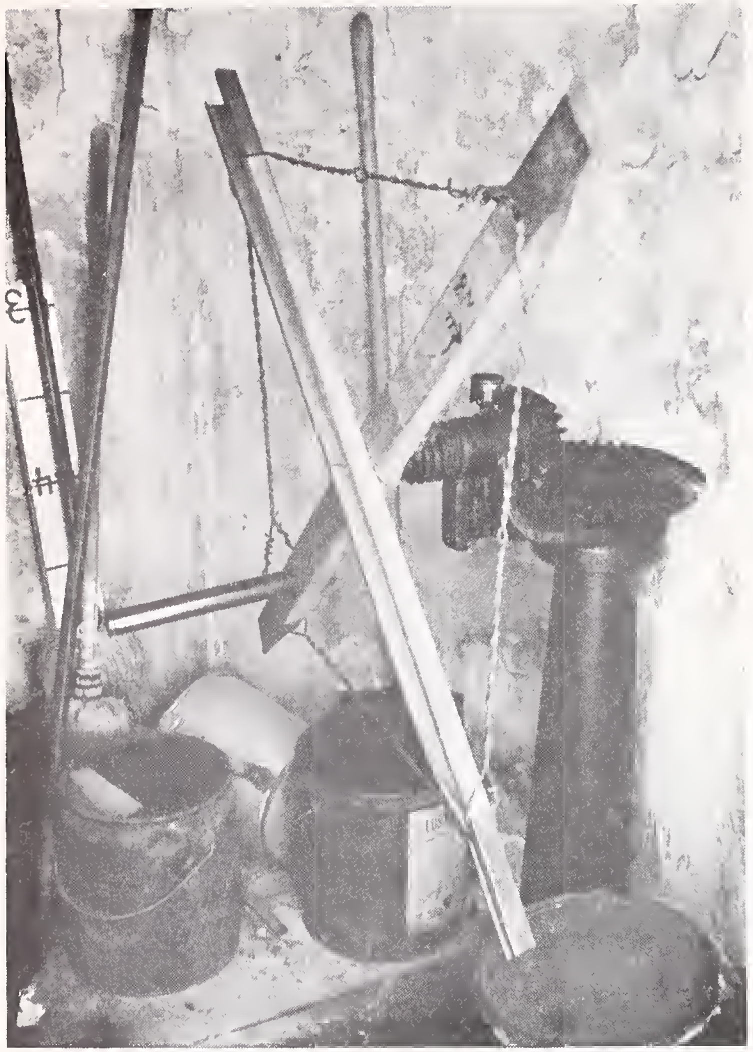


PHOTO 17 OUTLET VALVE CONTROLS

PHOTO 18 OUTLET VALVE CONTROL HOUSE

PHOTO 19 NOTE: OUTLET VALVE PLUG CENTER GROUND

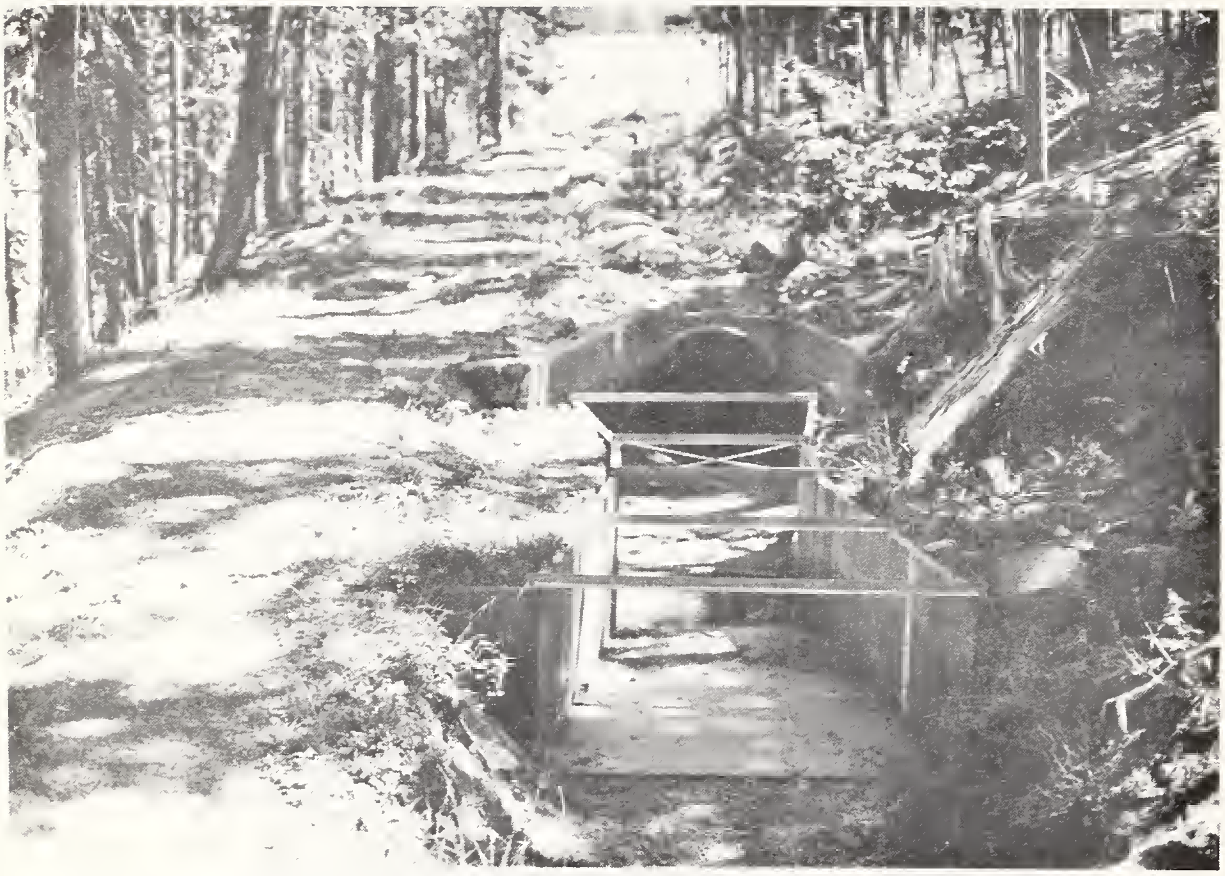


PHOTO 20 AQUEDUCT PIPE TO RESERVOIR

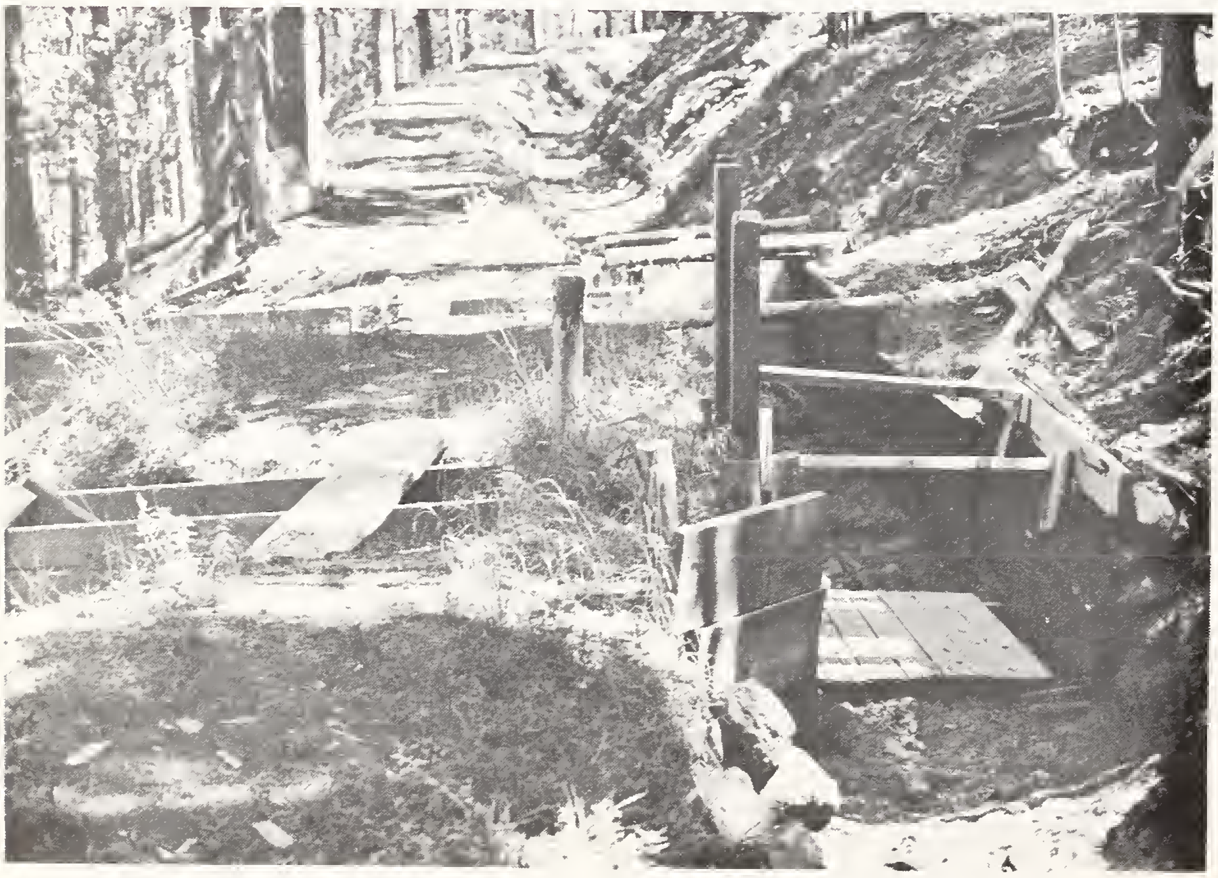


PHOTO 21 AQUEDUCT CHANNEL AND DIVERSION WORKS AHEAD OF PIPE

PHOTO 22 AQUEDUCT PIPE DISCHARGE TO RESERVOIR

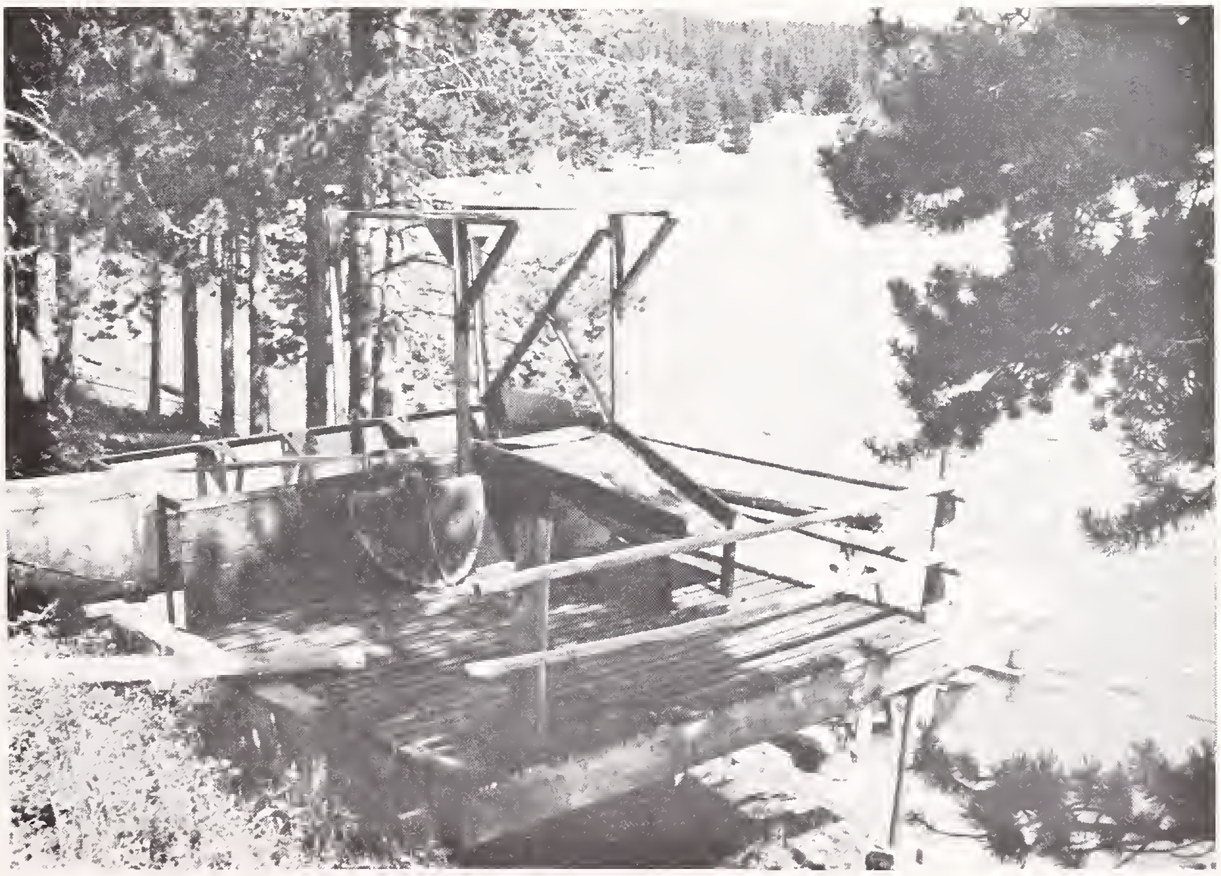


PHOTO 23 AQUEDUCT DISCHARGE WORKS

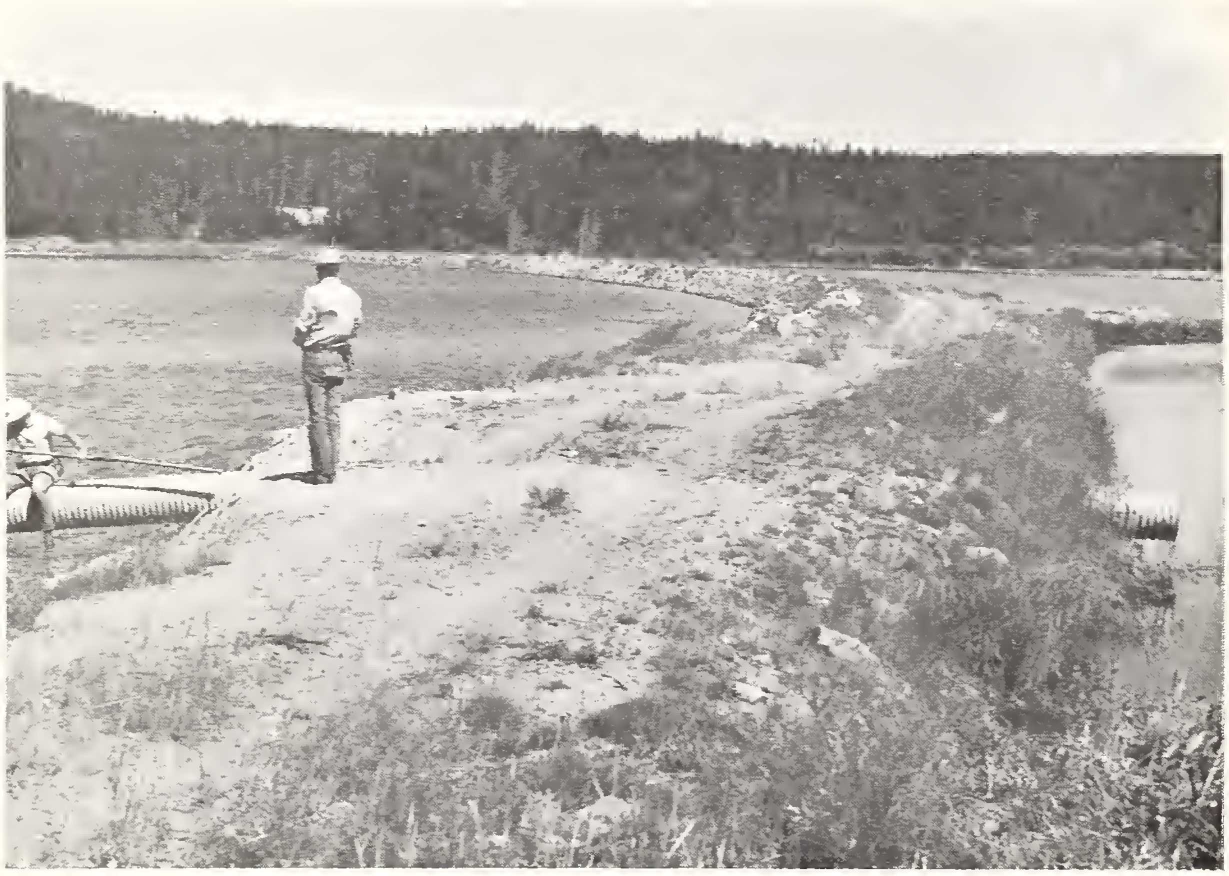


PHOTO 24 ROAD BERM ALONG SOUTH END OF RESERVOIR

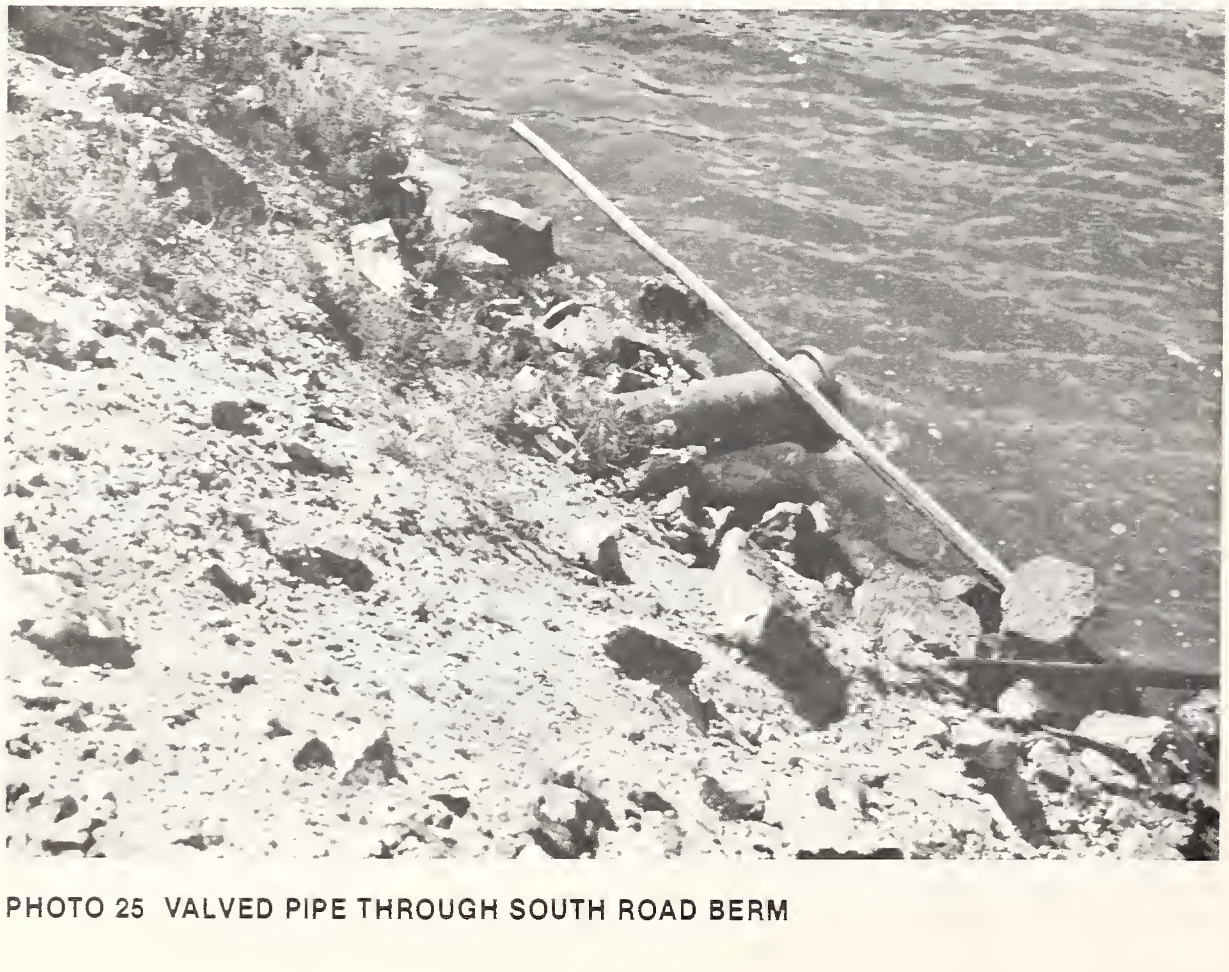


PHOTO 25 VALVED PIPE THROUGH SOUTH ROAD BERM

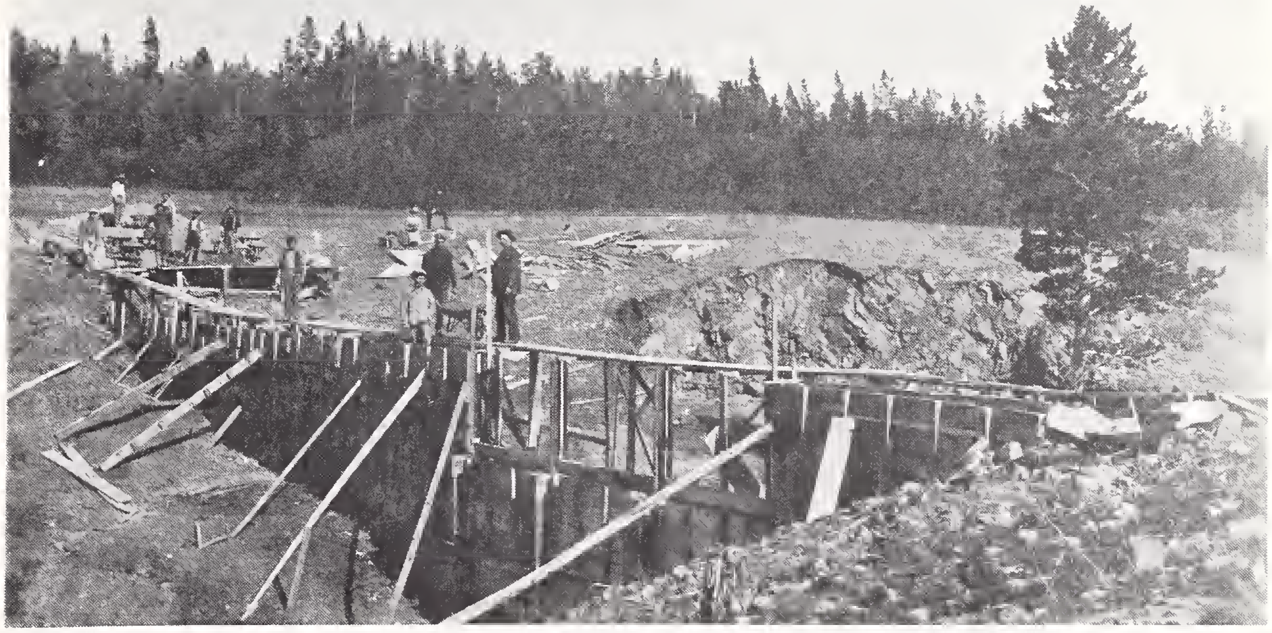


PHOTO 26 CHESSMAN SADDLE DAM DURING CONSTRUCTION. FORMING OF THE CORE WALL AND SPILLWAY GATES.
PHOTO COURTESY OF CITY OF HELENA

CHESSMAN SADDLE DAM

INSPECTION PHOTOGRAPHS TAKEN, AUGUST 9, 1978

PHOTO 28 APPROACH BASIN AND ROAD BERM UPSTREAM
OF SPILLWAY TAKEN FROM SPILLWAY

PHOTO 27 CHESSMAN SPILLWAY FROM THE TOE OF CHUTE

PHOTO 30 SPILLWAY APPROACH LOOKING TOWARD SOUTHWEST ABUTMENT

PHOTO 29 SPILLWAY AND APPROACH BASIN FROM ROAD BERM

PHOTO 32 SPILLWAY CHUTE AND DOWNSTREAM CHANNEL FROM SPILLWAY

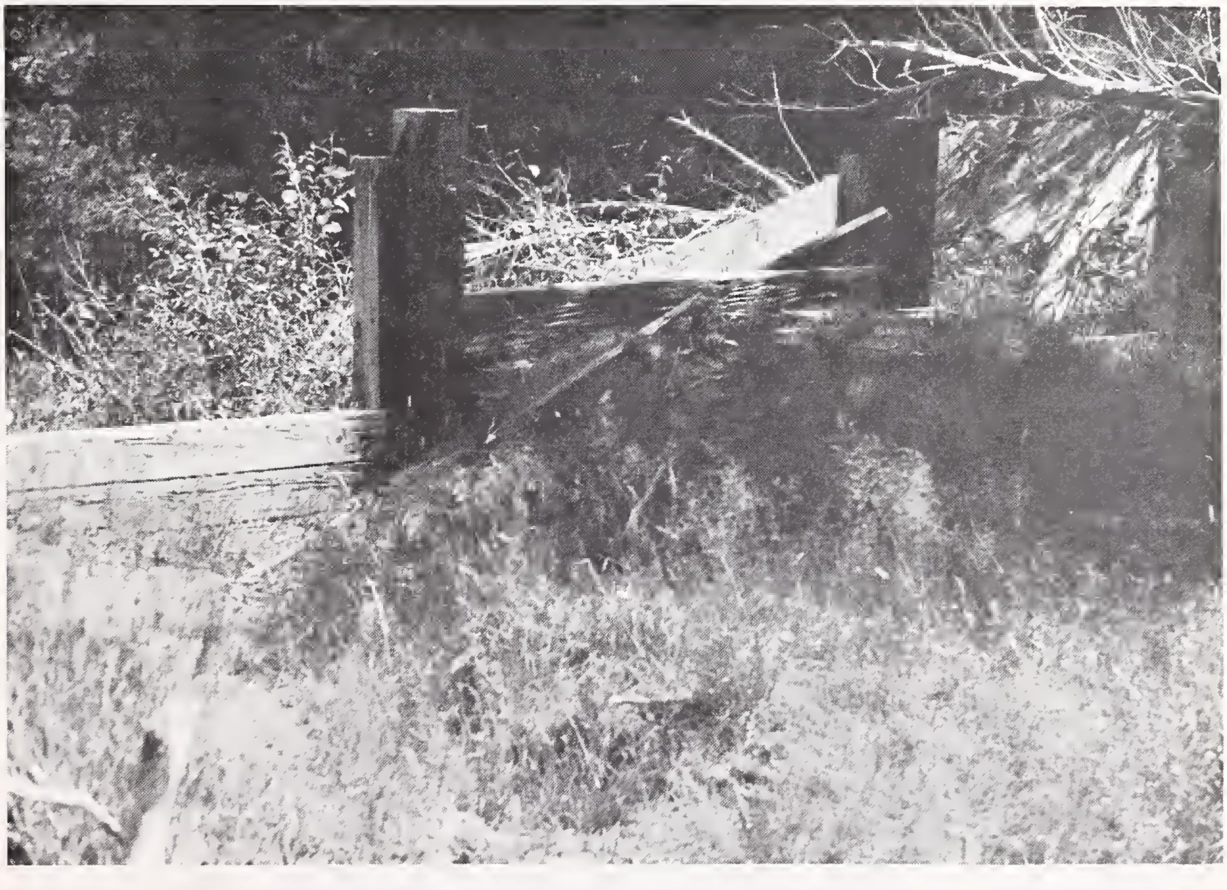


PHOTO 31 UPSTREAM SIDE OF SPILLWAY STOPLOGS

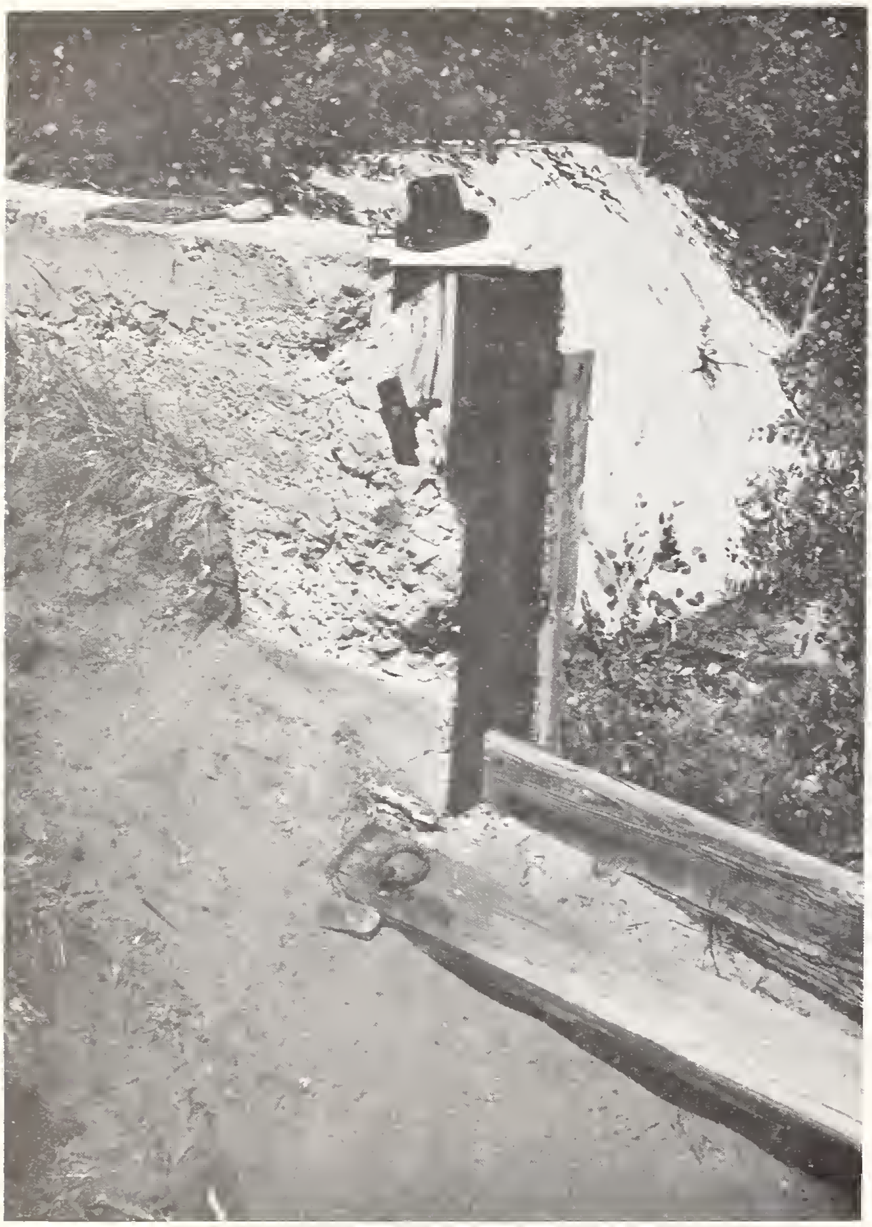


PHOTO 33 NORTHWEST GATE AND CORE WALL

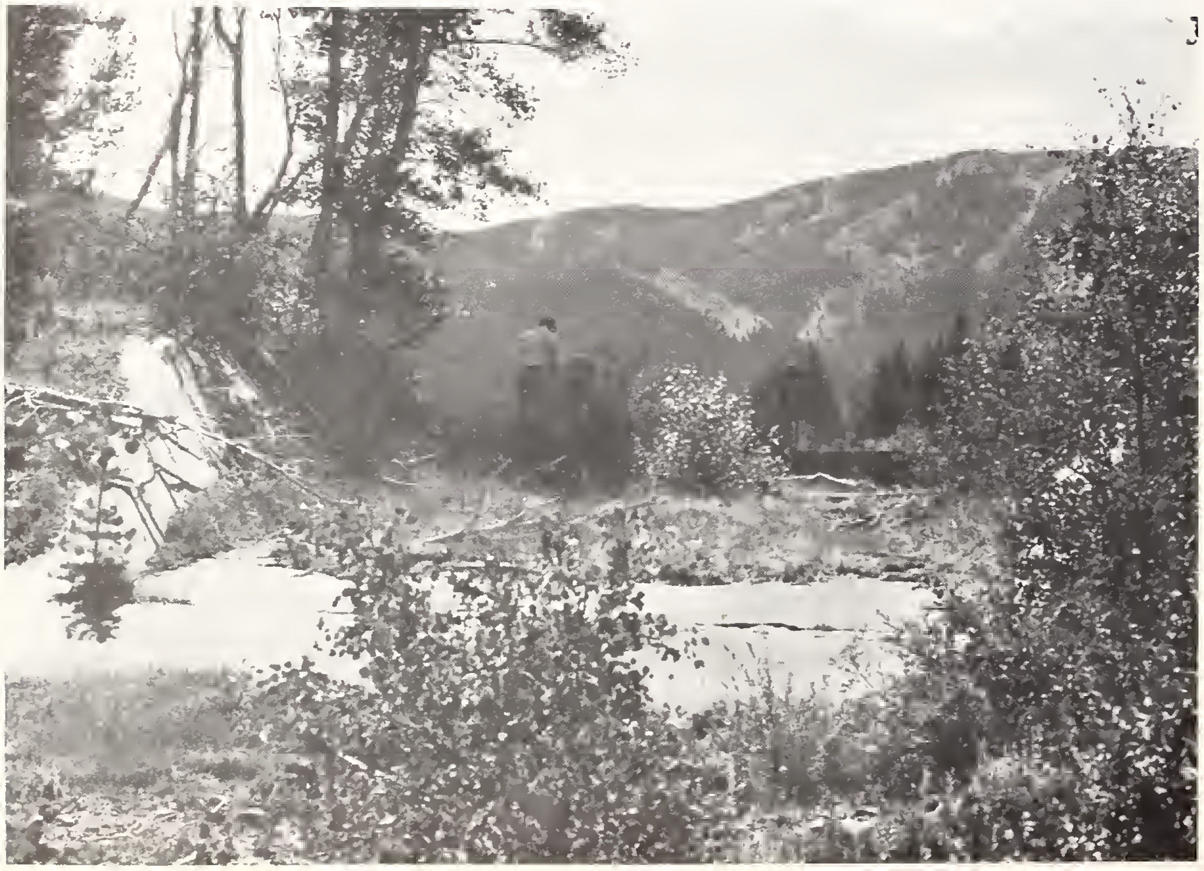


PHOTO 34 OVERGROWTH OF PAVED SPILLWAY CHUTE

PHOTO 36 PAVED CREST AND CORE WALL OF NORTHWEST SPILLWAY ABUTMENT LOOKING SOUTHEAST ALONG SPILLWAY AXIS

PHOTO 35
UPSTREAM FACE AND CORE WALL LOOKING SOUTHEAST ALONG AXIS OF SPILLWAY

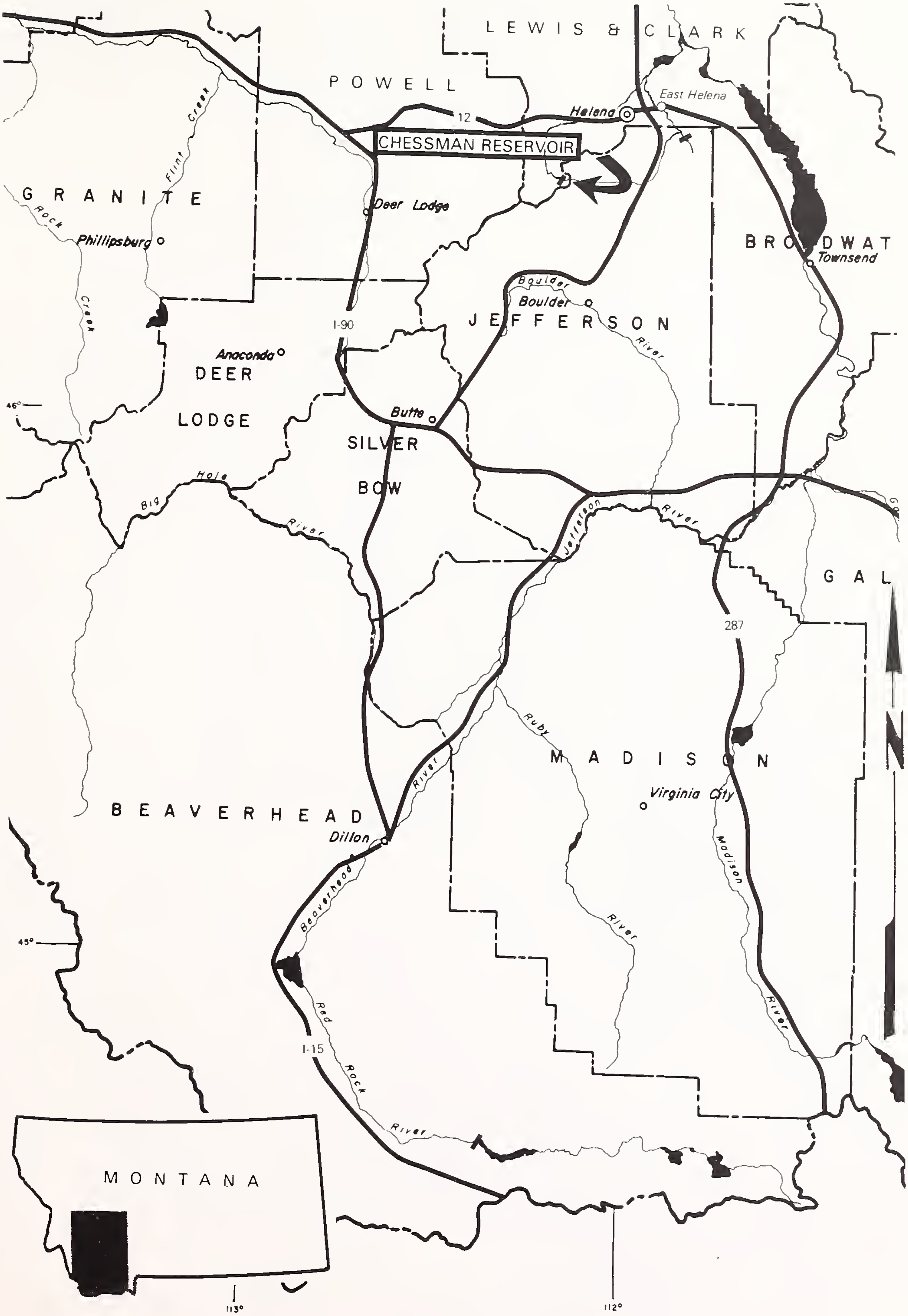
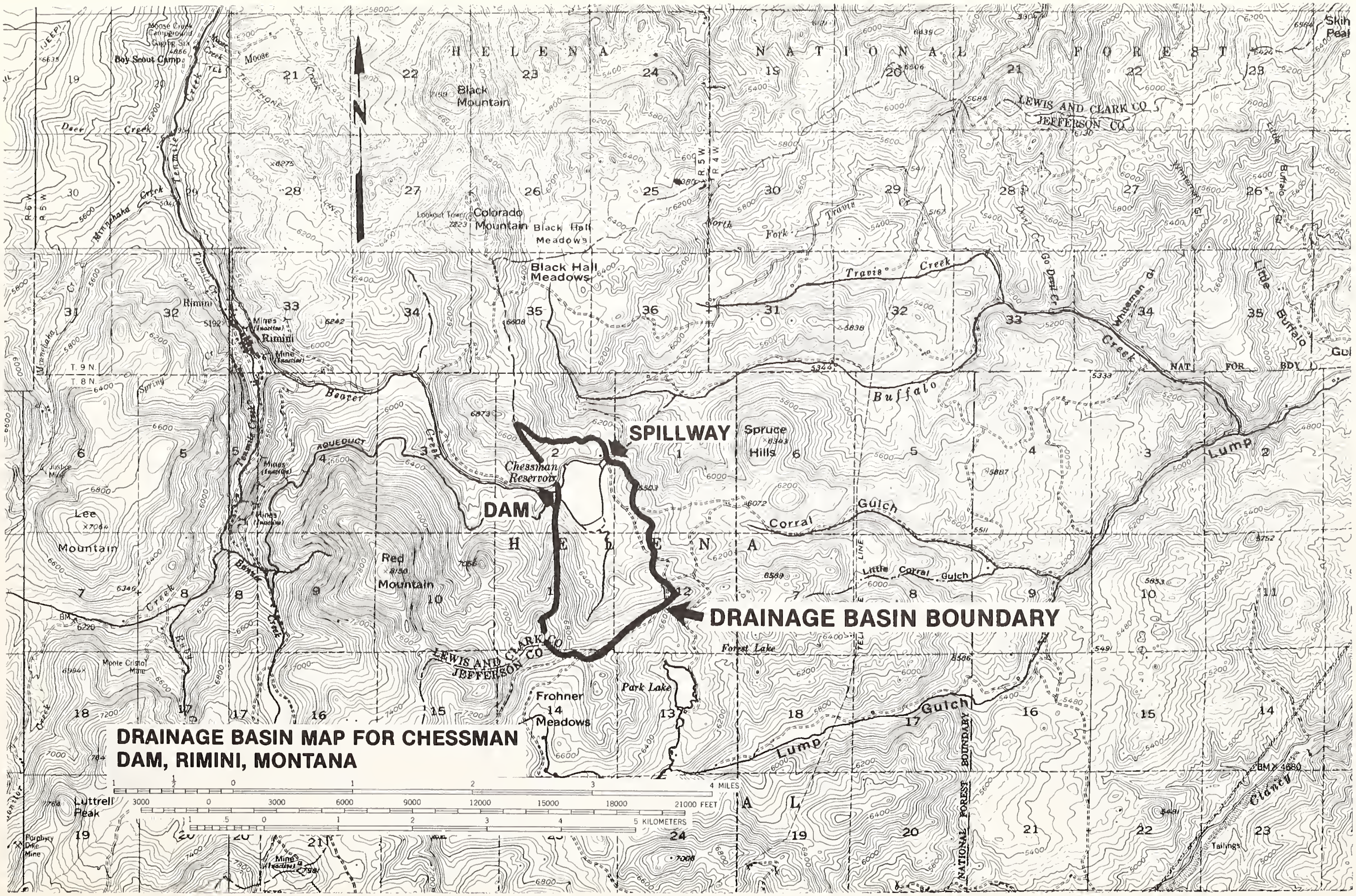
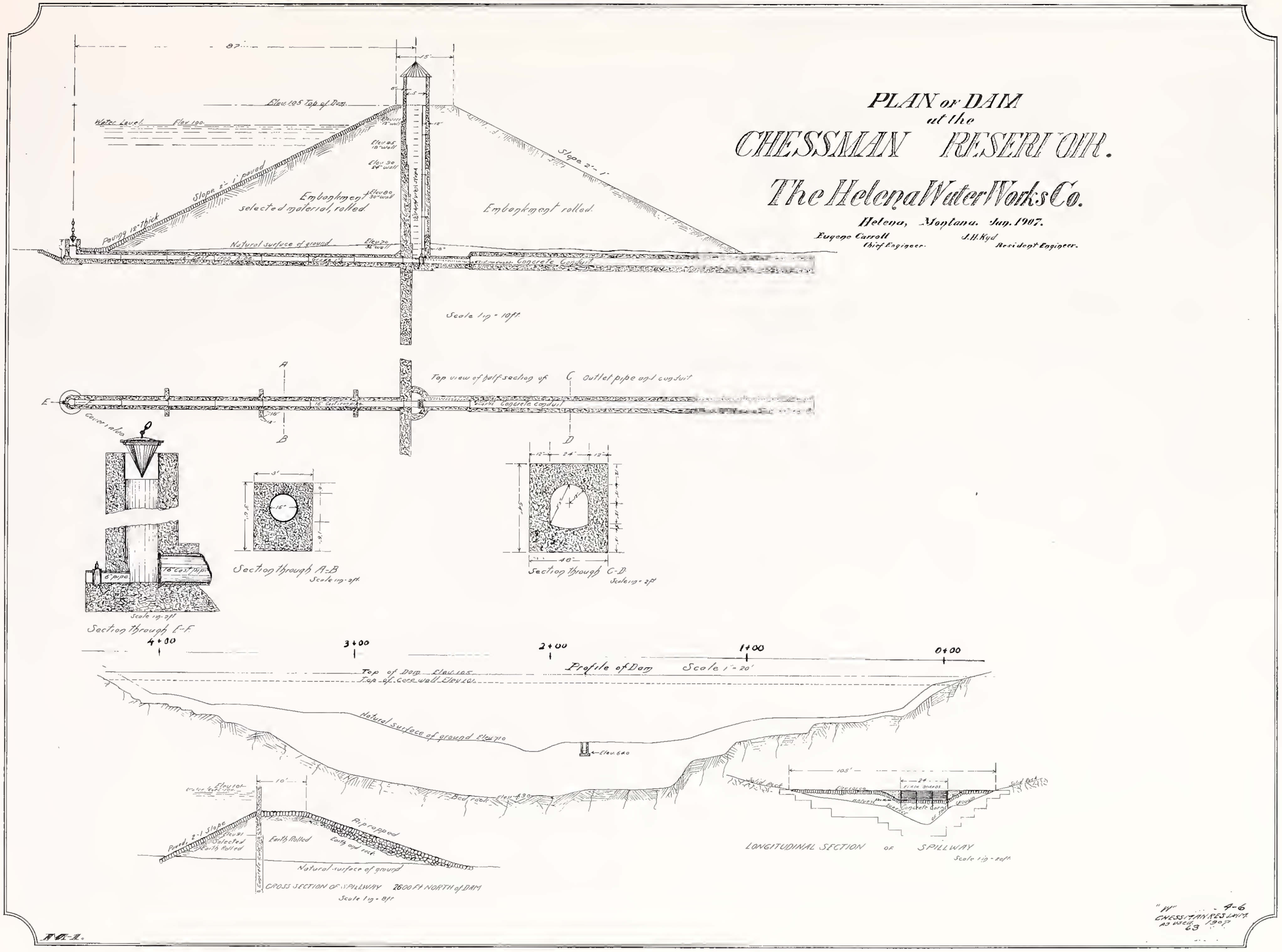


PLATE 1 CHESSMAN RESERVOIR DAM VICINITY MAP

PLATE 2

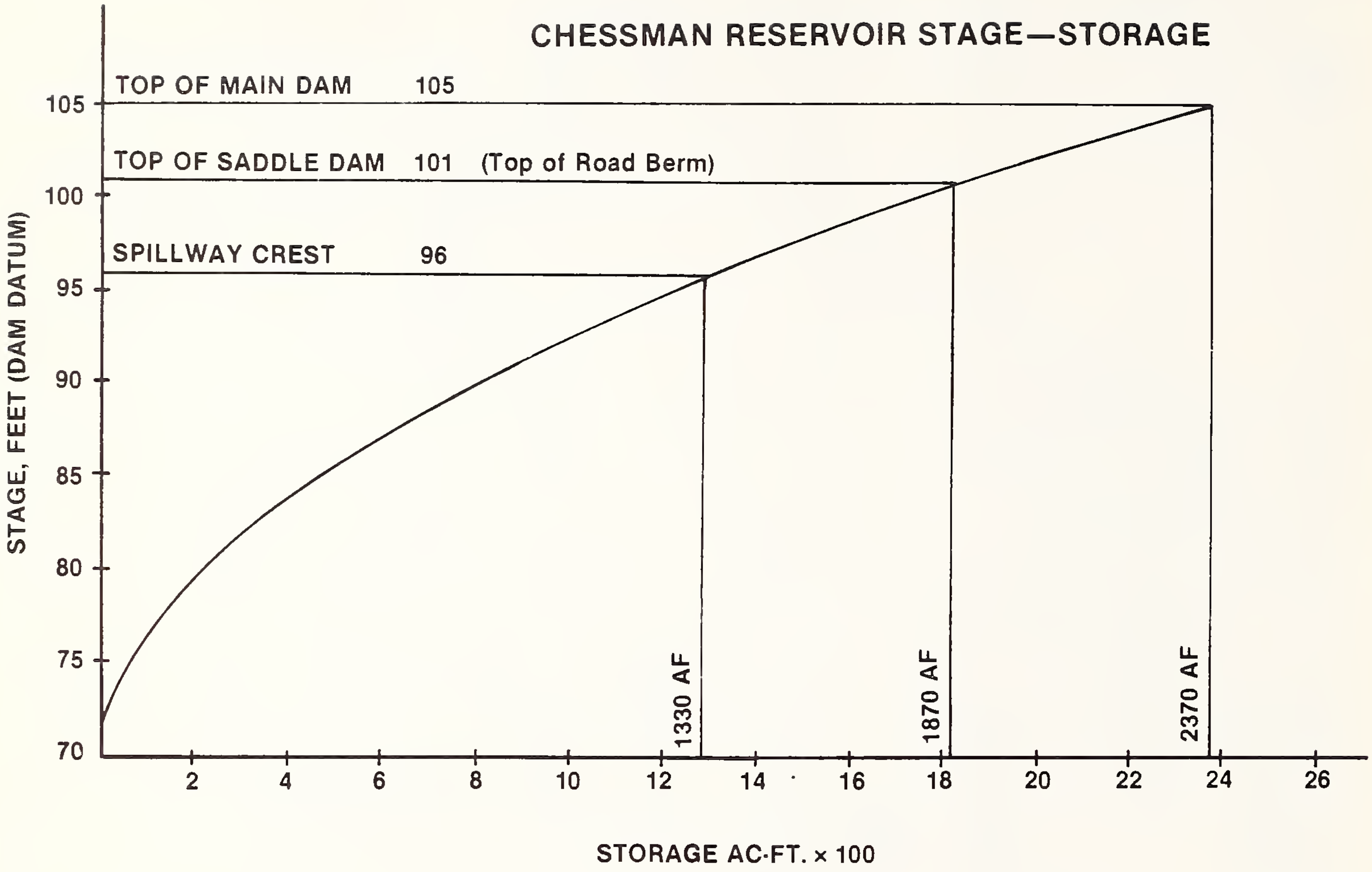


DRAINAGE BASIN MAP FOR CHESSMAN DAM, RIMINI, MONTANA



COURTESY OF CITY OF HELENA

PLATE 3


CHESSMAN RESERVOIR STAGE—STORAGE
TOP OF MAIN DAM 105
TOP OF SADDLE DAM 101 (Top of Road Berm)
SPILLWAY CREST 96
STAGE, FEET (DAM DATUM)
105
100
95
90
85
80
75
70
1330 AF
1870 AF
2370 AF
2
4
6
8
10
12
14
16
18
20
22
24
26
STORAGE AC·FT. × 100

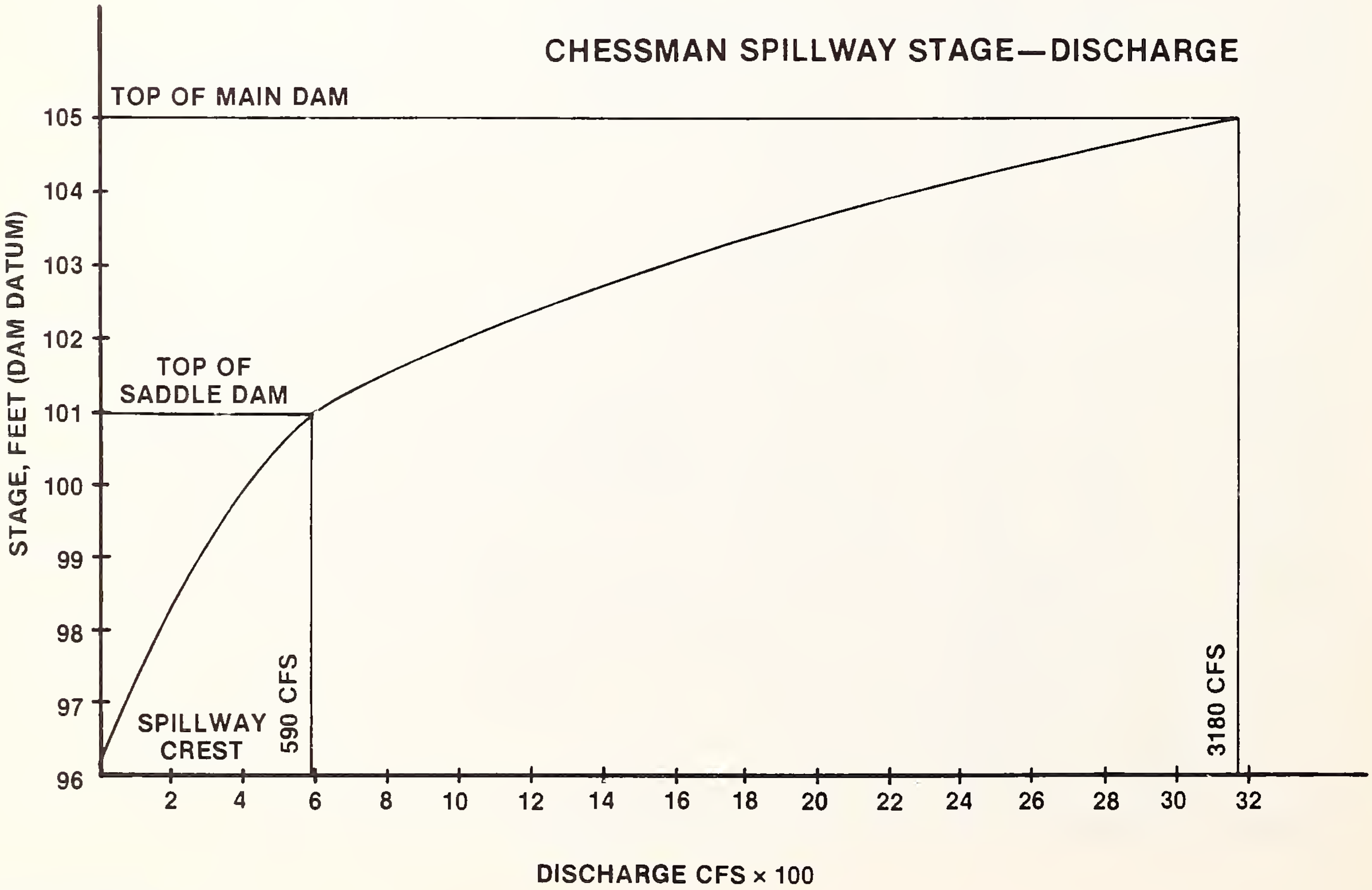
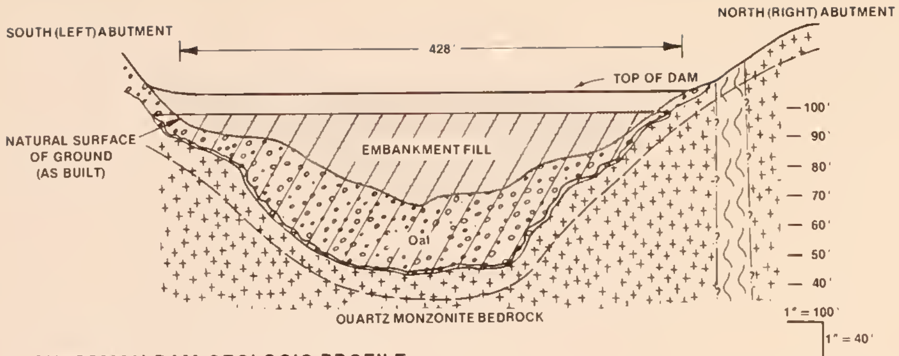

CHESSMAN SPILLWAY STAGE—DISCHARGE
TOP OF MAIN DAM
TOP OF SADDLE DAM
SPILLWAY CREST
STAGE, FEET (DAM DATUM)
105
104
103
102
101
100
99
98
97
96
590 CFS
3180 CFS
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
DISCHARGE CFS × 100

PLATE 4

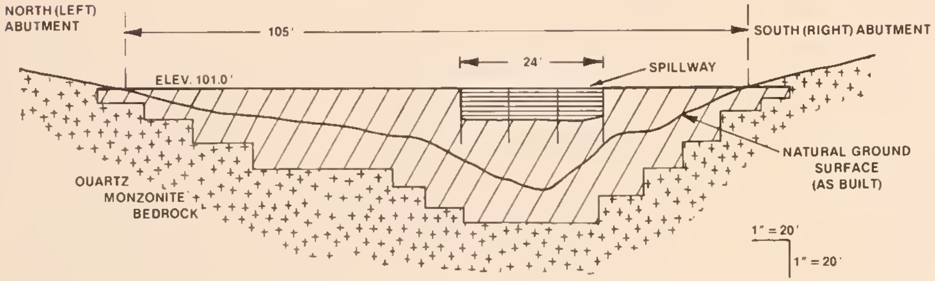


CHESSMAN DAM GEOLOGIC PROFILE

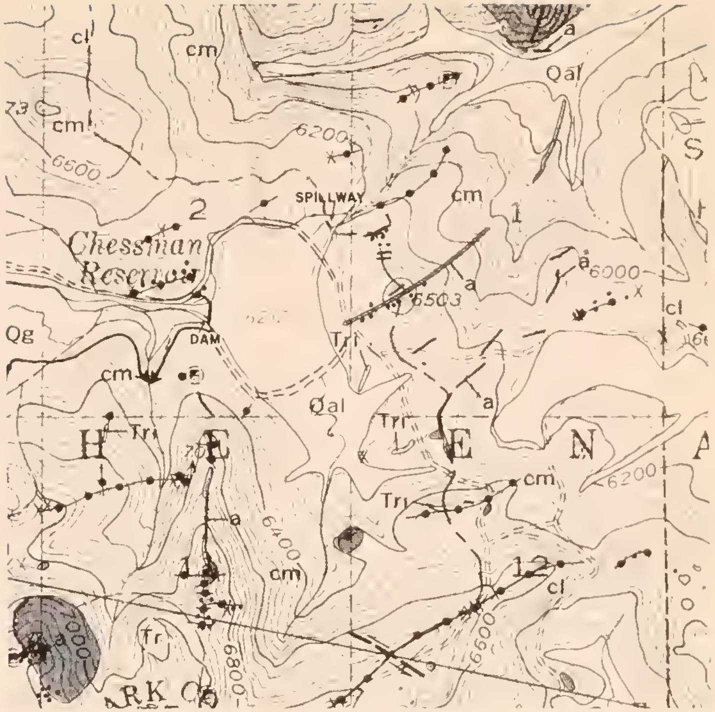
⦚ SHEAR ZONE

▨ COREWALL — TOP OF COREWALL ELEVATION AND CONTACT WITH BEDROCK CANNOT BE VERIFIED

— — POTENTIAL WEATHERED ZONE



CHESSMAN SADDLE DAM GEOLOGIC PROFILE



GEOLOGY FROM BECRAFT ET AL. (1963)

1 MILE

1" = 24,000'

EXPLANATION

Qal — QUATERNARY ALLUVIUM

Qg — QUATERNARY GLACIAL DEPOSITS

Tri — TERTIARY RHYOLITE INTRUSIVE ROCK

Tr — TERTIARY RHYOLITE INTRUSIVE AND FLOW ROCK

a — ALASKITE PORPHYRY, APLITE, AND PEGMATITE

cm cl ml — COARSE — AND MEDIUM GRAINED QUARTZ MONZONITE AND GRANODIORITE

—•—•—•— CHALCEDONY VEIN

⁘ HYDROTHERMALLY ALTERED ROCK

PLATE 5 GEOLOGY

CORRESPONDENCE

DEPARTMENT OF DEFENSE
UNITED STATES OF AMERICA

DEPARTMENT OF THE ARMY
SEATTLE DISTRICT, CORPS OF ENGINEERS
P.O. BOX C-3755
SEATTLE, WASHINGTON 98124

NPSEN-FM

18 OCT 1979

Richard A Nisbet, P.E.
Director of Public Utilities
Civic Center
Helena, Montana 59601

Dear Mr. Nisbet:

Thank you for your 23 August 1979 letter commenting on the draft dam safety report on Chessman Main and Saddle Dams.

We asked the Montana Department of Natural Resources and Conservation (DNR & C) to re-investigate the downstream hazard potential below the Saddle Dam as you requested. DNR & C and their engineering consultant have complied, and, although they both now believe the hazard potential should be significant rather than high, we believe further investigation is needed to accurately assess the downstream hazard potential below the Saddle Dam. Therefore, we are leaving the report essentially as written, except we are deleting the "unsafe" designation previously applied and are recommending that you, the owner, obtain the services of a competent consultant to perform a hydraulic routing of a Saddle Dam breach hydrograph to more accurately establish the potential for downstream damage and effects of dam failure. Utilizing results of this analysis, we are further recommending you and the DNR & C determine the appropriate spillway design flood from Tables 2 and 3, as presented in the inspection guidelines, and to then modify the project accordingly. If this study cannot be made, then we recommend the project be modified and/or operated to safely control the full PMF.

Because the Saddle Dam controls 1400 acre-feet of water, the size of the dam remains intermediate. In answer to your question relating to PMF frequency, we do not assign a frequency to the PMF.

We appreciate your interest in dam safety and the steps you are now undertaking in response to report recommendations.

Sincerely yours,

MAXEY B. CARPENTER, JR.
Lt. Colonel, Corps of Engineers
Acting District Engineer

Copy furnished:
DNR & C
Morrison-Maeirele

COMMISSIONERS
KATHLEEN RAMEY, MAYOR
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GARY L. DAVIS
RUSSELL J. RITTER

PUBLIC UTILITIES DIVISION
CIVIC CENTER
HELENA, MONTANA 59601
AREA CODE 406/442-9920

PUBLIC UTILITIES
DEPARTMENT

City of Helena, Montana

AUGUST 23, 1979

R.P. Sellevold, P.E.
Chief, Engineering Division
Dept. of the Army
Seattle District, Corps of Engineers
P.O. Box C-3755
Seattle, Washington 98124

Subject: Beaver Creek and Buffalo Creek
Chessman Dam and Chessman Saddle Dam
Near Rimini, Montana
MT-1090 and MT-1562

Dear Mr. Sellevold;

Reference is made to the final draft report on the city of Helena's Chessman Reservoir dam and Saddle Dam. I have reviewed the report and it appears to me that the conclusions are based on two primary factors; (1) the occurrence of a PMF (probable maximum flood) and, (2) such an occurrence would cause "severe downstream flooding, property destruction, and potential for loss of life as the surge travel down Buffalo Creek and Lump Gulch."

I can find no reference in the report to what frequency a PMF would occur. A comparison of the PMF volume and flood resulting from say, a 100 year storm would provide the city and the decision makers the comparative data needed to evaluate the total impact from the recommendation in the report.

The second factor is the assumption that "severe downstream flooding, property destruction and potential for loss of life as the surge traveled down Buffalo Creek and Lump Gulch" will occur if we experienced a PMF. Nowhere in the report can I find any detailed engineering data as to the actual capacity of Buffalo Creek and Lump Gulch and how it in fact relates to the PMF. Further, if the drainage in these two basins had been investigated, I contend one would find in fact no severe downstream flooding; minimum, if any, property destruction and no potential loss of life would occur from the hypothetical PMF.

The requirement for the city of Helena to lower the reservoir means that approximately 100 million gallon of storage will be lost to the city water supply. This requirement is based on the Saddle Dam being of intermediate size with a high downstream hazard. Actually, the dam is lower than the minimum height for a small size dam and the active storage impounded by the Saddle dam (approximately 1400 s.f.) is very close to the maximum allowable for a small dam. It does not seem unreasonalbe that the Saddle Dam be classed as a small dam.

The operational requirement then rests on the downstream hazard being high. Because of the very low density of development (3 structures in the first five miles downstream according to the USGS Quad Map), the high hazard classification is also marginal. We would request a more detailed analysis of the Buffalo Creek drainage be made so the hazard can be more definitely determined possibly resulting in reclassifying the dam as small with a significant hazard or less. This would greatly effect the operation of the reservoir.

In summary, it appears from the data in the report that a fairly detailed engineering analysis was done on the drainage basin contributing water to Chessman Reservoir and the evaluation of the dam and Saddle Dam and spillway. However, little or no detailed analysis was accomplished on the downstream effects from the PMF, but rather very general assumptions were made which have a significant effect on final recommendations. Based on this general data, you are in effect suggesting that the city of Helena reduce the amount of total storage by five feet which amounts to 29% of the total capacity of the reservoir. This is a very significant decrease in our total storage capacity of water for the city of Helena.

The city of Helena certainly does not intend to maintain a potential hazard if in fact one does exist. The city has taken the following steps as recommended in the study.

1. The water production superintendent has been instructed to develop a proposed operation and maintenance plan to provide for emergency and early downstream warning plan, routine maintenance and periodic safety inspection as recommended.

2. Plans are being made to restore the spillway to full serviceability.

3. The reservoir level was lowered to 96 feet project datum as of August 29, 1979.

4. We are reviewing the alternatives of removing the road berm across the spillway. Contacts will be made with the Forest Service to determine the most feasable alternative.

5. Engineering proposals will be requested from consulting engineers to provide a detailed PMF analysis, a full embankment seepage, and stability evaluation as recommended in the study.

It is our contention the above questions need to be resolved before the report is released to avoid any undue adverse publicity and for public emotional reactions based on incomplete data.

The city will be glad to meet with you and the Montana State Department of Natural Resources and Conservation at anytime to discuss in more detail our concerns.

Sincerely,

Richard A Nisbet

Richard A. Nisbet, P.E.
Director of Public Utilities

RAN/lmb

cc: Al Thelen, City Manager
Department of Natural Resources & Conservation

DEPARTMENT OF NATURAL RESOURCES & CONSERVATION
WATER RESOURCES DIVISION

STATE OF MONTANA

32 SOUTH EWING
HELENA, MT 59601

THOMAS L. JUDGE, GOVERNOR
TED J. DONEY, DIRECTOR

August 23, 1979

Ralph Morrison
Department of the Army
Seattle District, Corps of Engineers
P. O. Box C-3755
Seattle, Washington 98124

Dear Mr. Morrison:

The Department of Natural Resources and Conservation has reviewed the final draft report on the Chessman Dam and the Chessman Saddle Dam. We feel the report is complete and satisfies the criteria for the Phase I evaluation.

Thank you for this opportunity to review and comment on the final draft report for the Chessman Dam and the Chessman Saddle Dam.

Sincerely,

Richard L. Bondy

Richard L. Bondy, Chief
Engineering Bureau
(406) 449-2864

RLB/lh

UNITED STATES DEPARTMENT OF AGRICULTURE
FOREST SERVICE
FEDERAL BUILDING MISSOULA, MONTANA 59801

7530

AUG 9 1979

NPSEN-FM

R. P. Sellevold, P.E.
Chief, Engineering Division
Department of the Army
Seattle District, Corps of Engineers
P. O. Box C-3755
Seattle, Washington 98124

Dear Mr. Sellevold:

We have reviewed the final draft report of the Phase I inspection of the Chessman Reservoir Project and concur with your findings.

Included with the detailed hydrologic evaluation recommended, it seems a "breach dam analysis" should be performed under storm conditions and the resulting flood plain delineated. This would clarify the degree of hazard the structure presents and be an integral part of the "emergency action plan."

Sincerely,

R. W. LARSE
Regional Engineer

cc: Helena NF w/copy of Report

MORRISON-MAIERLE, INC.
QUALITY ASSURANCE
QA

Rodger C Foster
Project Manager

Thomas M Watson
Branch Manager or Department Head

Peer Reviewer

Harold L. Eagle
Principal-In-Charge

Chief Engineer No.	Date Approved
80-15	JUNE 17, 1980

Project No. 1447-006-01 (33)